Standard
Specifications for Fire Hydrants
for
Ordinary Water Works Service

ADOPTED BY AMERICAN WATER WORKS ASSOCIATION
JANUARY 17, 1940

EFFECTIVE FOR PURCHASES MADE AFTER NOVEMBER 1, 1940

The American Water Works Association has adopted and promulgates these basic "Standard Specifications for Fire Hydrants for Ordinary Water Works Service." They are based upon the best known experience and intended for use under normal conditions. They are not designed for unqualified use under all conditions and the advisability of use of the material herein specified for any installation must be subjected to review by the engineer responsible for the construction in the particular locality concerned.

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AMERICAN WATER WORKS ASSOCIATION

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These specifications supersede the "Standard Specifications for Fire Hydrants" adopted by A. W. W. A. June 24, 1913. They were published in the JOURNAL OF THE A. W. W. A. in April, 1937, as "Tentative Specifications"; revised June 10, 1937; August 10, 1938; and January 17, 1940; published in the JOURNAL OF THE A. W. W. A. as "Standard Specifications," August, 1940; and effective for purchases made on and after November 1, 1940.

Standard Specifications for Fire Hydrants

for

Ordinary Water Works Service

Section 1—Scope of Specifications

1.01 *Scope.* These specifications embrace the various types and classes of hydrants for general water-works service and cover the materials and workmanship employed in their fabrication. The setting and drainage of hydrants are not included in these specifications. For installation standards see "Standard Specifications for Laying Cast Iron Pipe," as adopted by the A. W. W. A. in 1938, or the latest revision thereof.

1.02 *Kind and Type.* The types of hydrants covered are the various forms of post hydrants with compression (opening against or with the pressure) or gate type of shut-off. High pressure, independently-gated and special hydrants are not included.

Section 2—Capacity and Size

2.01 *Capacity.* Capacity or delivery classification shall be designated according to the number of hose and pumper nozzles employed as follows: two-hose nozzle; one-pumper nozzle; one-hose and one-pumper nozzle; three-hose nozzle; two-hose and one-pumper nozzle; three-hose and one-pumper nozzle; four-hose nozzle; two-pumper nozzle, etc.

2.02 *Size.* The size of the hydrant shall be designated in terms of the minimum opening of the seat ring of the main valve and shall be stated in inches of diameter. With other than circular seat rings, the area of the seat ring opening shall equal that of a circular seat ring of equivalent size. Any diameter offered other than those enumerated shall be considered equal to the next lower size specified in this section.

The size of the hydrant (main valve opening) shall be at least four (4) inches for hydrants having two $2\frac{1}{2}$-inch nozzles; at least five (5) inches for hydrants having three $2\frac{1}{2}$-inch nozzles; and at least six (6) inches for hydrants having four $2\frac{1}{2}$-inch nozzles.

Combinations of nozzles, other than those above, are permissible,—provided that the total potential discharge capacity of such nozzle combination connected to the main valve does not exceed the maximum discharge capacity of nozzles connected to a main valve of the same size in the above list.

Thus, since it is understood that hose nozzles and pumper nozzles are not in action at the same time, one pumper nozzle may be added to any of the above hose nozzle combinations, or substituted for any one or two hose nozzles; provided that the discharge area of the pumper nozzle *either* (1) does not exceed the discharge area of the $2\frac{1}{2}$-inch hose nozzle or nozzles to which it is added or for which it is substituted *or* (2) does not exceed the discharge area of the main valve of the hydrant.

2.03 *Length.* The length of the hydrant shall be stipulated in feet (to the nearest half foot) to correspond to the depth of trench (in which it is to be installed) below the finished or established grade; or, expressed in other terms, it shall be the distance from the bottom of the connecting pipe to the ground line at the hydrant.

Section 3—Design

3.01 *Thickness of Wall of Barrel.* The thickness of the wall of the barrel shall be not less than the thickness specified for class 250 pit cast cast-iron water pipe of like diameter produced in accordance with "American Standard Specifications for Cast Iron Pit Cast Pipe for Water and Other Liquids" (ASA-A21.2-1939) and Table 3 thereof. Barrels of fractional inch diameters shall be made to the thickness of the next larger diameter listed below. The specified thicknesses thus are:

INSIDE DIAMETER OF BARREL IN INCHES	THICKNESS OF WALL IN INCHES
5	0.42[1]
6	0.43
7	0.46[1]
8	0.50
9	0.56[1]
10	0.63

Variations in thickness of walls of hydrant barrels shall not exceed those permitted for pit cast cast-iron pipe of like diameter as re-

[1] Values not in Table 3—but interpolated for purposes of these specifications.

corded in ASA Document A21.2 and Section 2-11(b) thereof, as follows:

NOMINAL DIAMETER IN INCHES INCLUSIVE	TOLERANCE, PLUS OR MINUS IN INCHES
3 to 8	0.07
10 to 24	0.08

Tolerances 0.02 inch greater than those above listed shall be permissible over areas not exceeding 8 inches in length in any direction. Such variations, if they exist, will be presumed to relate to inexact centering or buckling of the core at the time of pouring the iron and to imply a corresponding increase in thickness of the barrel at one point in the barrel to offset the decrease in thickness at the point diametrically opposite.

3.02 *Barrel Sections.* If hydrants are made in two or more sections with a flange or other joint near the ground line, the joint shall, unless otherwise specified by the purchaser, be located at least two inches above the finished grade line.

3.03 *Waterway.* Changes in shape or size of the waterway shall be accomplished by means of easy curves. The junctions of hose and pumper nozzles with the barrel shall be rounded to ample radii. Exclusive of the main valve opening, the net area of the waterway of the barrel and foot-piece at the smallest part shall be not less than 120 per cent of that of the net opening of the main valve.

3.04 *Hydrant Inlet.* The base of the hydrant, known as the foot-piece or elbow, shall have a side or bottom inlet, provided with a bell, a flange, or other type of connection as specified or approved by the purchaser for connecting the hydrant to the branch from the main. The inlet shall have a clear waterway of not less than 6 inches nominal diameter unless otherwise ordered. In a hydrant provided with bell type of connection, the bell dimensions shall conform to those shown in the most recent Standard Specifications for Cast-Iron Pipe and Special Castings as adopted by the American Water Works Association. In a hydrant provided with flange type connections, the flange dimensions shall conform to A.S.A. (125-pound) Standard (B16a—1939).

3.05 *Lugs.* When so ordered by the purchaser, lugs for harnessing the hydrant to the connecting pipe from the street main shall be provided on the bell of the elbow.

3.06 *Hose Threads.* Unless otherwise specified by the purchaser the hose threads on the hose and the pumper nozzles shall conform

to the requirements of A.S.A. Specifications (B-26-1925) for the "National (American) Standard Fire Hose Coupling Screw Thread for all connections having nominal inside diameters of $2\frac{1}{2}$, 3, $3\frac{1}{2}$ and $4\frac{1}{2}$ inches," or the latest revision thereof.

3.07 *Joining Nozzles to Barrel.* Hose nozzles must be of composition metal and fastened into the barrel by a fine thread or by leading. If lead is used, an adequate recess shall be provided for the lead. All nozzles shall be safeguarded against blowing out or in the case of the screwed-in nozzles, a pin or other approved method shall be employed to prevent the nozzle turning or backing out.

3.08 *Nozzle Caps.* Nozzle caps shall be provided for all outlets. The threads shall conform to those of the nozzle. The cap nut shall have dimensions similar to those of the operating nut. Unless otherwise ordered caps shall be securely chained to the barrel with a metal chain having links not less than $\frac{1}{8}$ inch in diameter, or of equivalent cross sectional area. A recess shall be provided at the inner end of the threads to retain a gasket. When specified by the purchaser, each cap shall be provided with a suitable gasket to secure a tight seal with the nozzle.

3.09 *Valves Readily Removable.* The hydrant shall be so designed that when in place no excavation will be required to remove the main valve and the movable parts of the drain valve.

3.10 *Main Valve Remains Closed After Accident.* The barrel and operating mechanism shall be so designed that in case of accident, damage or breaking of the hydrant above or near the grade level, the main valve will remain closed reasonably tight against leakage or flooding.

3.11 *Operating Mechanism.* The operating threads of the hydrant shall be designed so as to avoid the working of any iron or steel parts against other iron or steel. Either the operating stem or the operating nut shall be, or both may be, of non-corrodible metal. The operating stem and nuts shall have square, V, or Acme threads which shall be designed with a factor of safety of not less than 5 for all stresses that may occur when a wrench with a 15-inch arm is used on a hydrant subjected to water pressure of 150 pounds per square inch.

3.12 *Stuffing Boxes.* Stuffing boxes shall be made of cast iron or bronze. The width of the packing shall be at least $\frac{1}{4}$ inch, and the depth of the packing space shall be at least four times its width. Glands shall be made of solid bronze, or of cast iron with bronze bushings. If a packing nut is used it shall be of bronze or other non-corrodible metal, and shall be properly secured so that it will not revolve with the operating stem. Gland bolts or studs may be

iron, steel or non-corrodible metal. If of non-corrodible metal, their diameter shall be not less than $\frac{5}{8}$ inch; or, if of other metal, not less than $\frac{1}{2}$ inch. The nuts shall be made of bronze or other suitable non-corrodible metal.

3.13 *Drain Valve Mechanism.* A positive operating non-corrodible drain valve or valves shall be provided to drain the hydrant properly by opening as soon as the main valve is closed, and to close tightly when the main valve is opened. The seat of the drain valve shall be made of bronze or other non-corrodible material and shall be fastened securely in the hydrant.

3.14 *Drain or Waste Outlet.* An outlet for drainage shall be provided in the base or barrel, or between the base and barrel of the hydrant. It shall be made of bronze or non-corrodible metal, or bushed therewith completely from the valve to the outside. When stipulated by the user, the opening shall be tapped for receiving a threaded pipe to carry the drainage away from the hydrant.

3.15 *Hydrant Top.* The hydrant top or bonnet shall be free-draining, and of a type that will maintain the operating mechanism in readiness to use under freezing condition. It shall be so designed to make tampering difficult, and be provided with convenient means to afford lubrication to insure ease of operation, and the prevention of wear and corrosion.

3.16 *Operating and Cap Nuts.* The operating nut and the cap nuts on the hose and pumper nozzles shall conform to those in service in the system where the hydrant is to be installed. Unless otherwise required, nuts shall be of pentagonal shape. The pentagon shall measure $1\frac{1}{2}$ inches from point to flat at the base of the nut and $1\frac{7}{16}$ inches at the top. Faces shall be tapered uniformly, and the height of the nut shall be not less than 1 inch. The point to flat dimension is to be measured to the theoretical point where the faces would intersect were there no rounding off of the corner. Wrenches for these nuts shall have no taper in the openings, so as to be readily reversible.

3.17 *Direction of Opening.* The direction of rotation of the operating nut to open the hydrant should conform to the practice in the water system where the hydrant is to be installed. Unless otherwise ordered by the purchaser, the hydrant shall be opened by turning the operating nut to the left, counter clockwise. An arrow and the word "OPEN" shall be cast in relief to be clearly visible, on the top of the hydrant to designate the direction of opening.

3.18 *Interchangeability.* Similar parts of hydrants of the same model, size, and make shall be interchangeable with each other without any special fitting.

3.19 *Machined Surfaces.* All machined surfaces shall be so designated by the manufacturer on any drawings requested by the user for approval. Such surface shall be suitably finished for the service and fitting intended.

3.20 *Water Hammer.* The operating mechanism, particularly the pitch of the thread of the operating stem, shall be so designed that when the operating nut is turned at a proper rate to shut off the flow of water, the static pressure plus water hammer shall not exceed twice the static—if the static pressure averages 60 pounds per square inch or greater. If the static pressure averages less than 60 pounds per square inch the pressure shall not be raised more than 60 pounds above static.

3.21 *Friction Losses.* With a hydrant 5 feet in length discharging 250 gallons per minute through each $2\frac{1}{2}$-inch hose outlet, the friction loss shall not exceed the following:

FLOW	PRESSURE LOSS
gallons per minute	*pounds per square inch*
250	1
500	2
750	3
1000	4

Section 4—Materials of Construction

4.01 *Parts of Cast Iron.* The parts where cast iron may be used are the foot-piece or elbow, the barrel or standpipe, the frost jacket if used, the bonnet, the gates of gate-type hydrants, the nozzle caps and small miscellaneous parts where the use of cast iron will conform to good practice.

4.02 *Cast Iron.* All cast-iron parts shall be made of a superior quality of iron. Cast iron shall conform to Specification A-126-30 for Class B Gray Iron Castings for Valves of the American Society for Testing Materials, or the latest revision thereof. All castings shall be clean and sound, without defects of any kind. No plugging, welding or repairing of defects will be permitted.

4.03 *Wrought Iron.* Wrought iron, where used, shall conform to Specification A-41-36 or A-84-36 of the American Society for Testing Materials, or the latest revision thereof.

4.04 *Steel.* Steel, where used, shall conform to Specifications A-31-36 or A-107-36 or A-18-30 of the American Society for Testing Materials or the latest revision thereof.

4.05 *Parts of Non-Corrodible Metal.* The parts where bronze or other non-corrodible metal shall be used are:—the hose and pumper nozzles; the valve seat or seat ring; the drain valve mechanism; and other parts elsewhere herein so specified or where the use of non-corrodible metal will conform to good practice.

4.06 *Characteristics of Non-Corrodible Metal.* All parts made of a non-corrodible alloy such as brass or bronze shall be made from a metal having a tensile strength of not less than 30,000 pounds per square inch, a yield point of not less than 14,000 pounds per square inch, and an elongation of not less than fifteen per cent (15 per cent) in two inches.

4.07 *Facing of Main Valve Against Seats.* The main valve of the hydrant shall be faced with a suitable yielding material such as rubber, leather, balata or composition where it bears on metal seats. The material shall be clamped so that the valve will not leak at the stem. The bottom stem threads may be protected by a suitable cap nut.

4.08 *Bolts and Nuts.* All bolts and nuts shall conform to Specification B18.2-1933 of the American Standards Association, or the latest revision thereof. They shall be of first class workmanship and the metal shall conform in physical characteristics to the provisions of these specifications for wrought iron or wrought steel or bronze, respectively. Unless otherwise specified by the purchaser, special treatments such as Parkerizing to minimize corrosion will not be applied.

Section 5—Details of Construction

5.01 *Test Bars.* At least one test bar shall be made and tested by the manufacturer from each heat of metal used, in accordance with the Specifications of the American Society for Testing Materials pertaining thereto.

5.02 *Shop Inspection.* Representatives (such as the engineer or inspecting engineer) of the purchaser shall be given at all times ready access to places where material for the purchaser is being produced, and they shall have authority to pass on quality and workmanship, and to disapprove and reject work and materials which in their judgment do not conform to the spirit and intent of these specifications.

5.03 *Shop Tests.* Hydrants, for a working pressure of 150 pounds per square inch or less, shall be subjected, after assembly, to two shop tests under a hydraulic pressure of 300 pounds per square inch. One test shall be made with the whole interior of the hydrant under

pressure; and another with the main valve closed and the foot-piece under pressure from the inlet side. Unsatisfactory conditions due to leakage or other imperfections found in either test shall be corrected before the hydrant is accepted.

Section 6—Marking

6.01 *Marking.* All hydrants shall have permanent markings, identifying the manufacturer by name, initials or abbreviations in common usage, designating the size of the main valve opening and the year of manufacture. Markings shall be so placed as to be readily discernible and legible after hydrants have been installed.

Section 7—Painting and Coating

7.01 *Coating.* All iron parts of the hydrant, inside and outside, shall be thoroughly cleaned and thereafter, unless otherwise stipulated, all surfaces except the exterior portion above the ground line shall be coated, painted with or dipped in an asphalt or bituminous base paint or coating. If these parts are painted they shall be covered with two coats, the first being allowed to dry thoroughly before the second is applied. If these parts are dipped they shall be preheated and the coating material shall be heated to a temperature employed in the coating of cast-iron pipe.

7.02 *Shop Coating Above Ground Line.* The outside of the hydrant above the finished ground line shall be thoroughly cleaned and thereafter painted in the shop with two coats of paint of a durable and weatherproof composition which shall produce a surface to which later coats with a linseed oil or other approved base will readily adhere. The color or colors to be used shall be as specified by the purchaser.

Supplementary Information

In purchasing hydrants under these specifications ("A. W. W. A.—1940") it will be necessary to make specific statements listing the requirements with relation to the following details:

	Refer to Paragraphs
Size of main valve opening	2.02
Depth of trench (length of hydrant)	2.03
Diameter of inlet connection	3.04
Style of inlet connection—bell or flange	3.04
Number and size of hose nozzles	2.03 and 3.06
If A.S.A. (National) Standard Thread, so specify	3.06
If not A.S.A. (National) Standard Thread, specify the following details:	
Outside diameter of male thread finished.	
Diameter of root of male thread.	
Number of threads per inch.	
Pattern of thread.	
Cut off——of an inch at top of thread.	
Leave —— of an inch in bottom of valley.	
Number and size of pumper nozzles	2.03 and 3.06
If A.S.A. (National) Standard Thread, so specify	3.06
If not A.S.A. (National) Standard Thread, specify the following details:	
Outside diameter of male thread finished.	
Diameter of root of male thread.	
Number of threads per inch.	
Pattern of thread.	
Cut off —— of an inch at top of thread.	
Leave —— of an inch in bottom of valley.	
Harnessing lugs, if required	3.05
Nozzle cap washers, if required	3.08
Shape and size of operating nut	3.16
Direction to turn to open	3.17
Corrosion resistance treatment, if required	4.08
Color above ground	7.02

Tentative

Standard Specifications

for

Elevated Steel Water Tanks, Standpipes and Reservoirs

American Water Works Association

American Welding Society

—1940—

The American Water Works Association has adopted and promulgates these "Tentative Standard Specifications for Elevated Steel Water Tanks, Standpipes and Reservoirs." They are based upon the best known experience and are intended for use under normal conditions. They are not designed for unqualified use under all conditions and the advisability of use of the material herein specified for any installation must be subjected to review by the engineer responsible for the construction in the particular locality concerned.

First Printing, December, 1940

AMERICAN WATER WORKS ASSOCIATION

22 East 40th St., New York, N. Y.

Foreword

These specifications were prepared by a joint committee of the American Water Works Association and the American Welding Society consisting of L. R. Howson, General Chairman, H. O. Hill, H. A. Sweet, C. W. Obert, J. P. Schwada, J. O. Jackson, H. C. Boardman and N. T. Veatch, Jr.

These specifications are a revision and extension to include welded construction of the specifications entitled "Standard Specifications for Riveted Steel Elevated Tanks and Standpipes" which were prepared by Subcommittee 7H of the American Water Works Association and published tentatively in the JOURNAL in December, 1931, and finally in the JOURNAL, November, 1935. Those specifications covered riveted construction only. The purpose of the present specifications is to provide a uniform guide for minimum requirements as to the design, fabrication and erection of elevated steel water tank, standpipe and reservoir structures of either welded or riveted construction.

All requirements relative to welding in these specifications have been taken from American Welding Society "Tentative Rules for Field Welding of Storage Tanks (1940)."

These specifications have been approved as "Tentative Standard" by the Executive Committee of the American Welding Society, April 4, 1940; and by the Board of Directors of the American Water Works Association, April 25, 1940.

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SECTION

Tentative

Standard Specifications

for

Elevated Steel Water Tanks, Standpipes and Reservoirs

Section 1—General

1.01 *Local Requirements.* Where local, municipal, county, state or government requirements exist such requirements are to govern and these specifications shall be interpreted to supplement them.

1.02 *Definitions.* Under these specifications the following definitions will apply:

Elevated Tank shall mean a container or storage tank supported on a tower.

Standpipe shall mean a flat bottom cylindrical tank having a shell height greater than its diameter.

Reservoir shall mean a flat bottom cylindrical tank having a shell height equal to or smaller than the tank diameter.

Tank shall mean an elevated tank, a standpipe or a reservoir.

Purchaser shall mean the person, company or organization which purchases the tank.

Engineer shall refer to the purchaser's engineer.

Contractor shall mean the person or company who contracts with the purchaser under these specifications to furnish and erect the tank.

Capacity shall mean that contained between the level of the overflow and the lowest specified level.

1.03 *What the Purchaser Is to Furnish.* The purchaser shall furnish the site upon which the tank is to be built with sufficient space to permit the contractor to erect the structure using customary methods. The purchaser shall furnish foundations. Unless otherwise agreed the purchaser shall furnish at the tank site the water at the proper pressure for testing and facilities for disposing of waste water, after testing. The purchaser shall furnish a suitable right of way from the nearest public road to the erection site.

1.04 *What the Contractor Is to Furnish.* The contractor shall furnish foundation plans, the anchor bolts, all materials except for foundations, all labor necessary to complete the structure including

the accessories required by these specifications and any additional work or accessories separately specified by the purchaser.

1.05 *Information to Be Furnished by Purchaser.* In his advertisement or inquiry the purchaser shall furnish the information itemized in Section A-2 for elevated tanks or that in Section A-3 for standpipes or reservoirs.

1.06 *Information to Be Furnished by Bidder.* Each bidder shall furnish the information itemized in Section A-4 for elevated tanks or that in Section A-5 for standpipes or reservoirs.

1.07 *Guarantee.* The contractor shall guarantee the structure against any defective materials or workmanship including paint and painting if in accordance with Sections A-8, A-9 and A-10 for a period of one year from date of completion. If any materials or workmanship prove to be defective within one year they shall be replaced or repaired by the contractor.

Section 2—Materials

2.01 *Bolts, Anchor Bolts and Threaded Rods.* Bolts, anchor bolts and threaded rods shall conform with any of the following A.S.T.M. Specifications: A 7, Bolting Material; A 10; A 78 (for welded construction specify for fusion welding); or A 141, latest revision.

For welded construction the carbon content should not exceed 0.30 per cent nor the manganese 1.00 per cent (by ladle analysis).

2.02 *Reinforcing Steel.* Reinforcing steel shall comply with the latest revision of A.S.T.M. Specifications A 15 (structural or intermediate grade).

2.03 *Plates.* Plate materials shall conform to the latest revision of any of the following A.S.T.M. Specifications A 7, A 10, A 78 (for welded construction specify for fusion welding), A 89 (for welded construction specify for fusion welding) or A 113 (cold pressing grade).

For welded construction the carbon content shall not exceed 0.30 per cent, nor the manganese content 1.00 per cent (by ladle analysis).

2.04 *Basis of Furnishing Plates.* Plates shall be furnished to average weight per square foot with permissible underrun and overrun according to the tolerance table for plates ordered to weight published in the applicable A.S.T.M. specification. Plate thicknesses shown on purchaser's or contractor's plans or specifications shall be converted to average weight per square foot by multiplying the thickness in inches by 40.8.

2.05 *Structural Shapes.* Structural materials shall conform to the latest revision of A.S.T.M. Specifications A 7 or A 10.

For welded construction the carbon content shall not exceed 0.30 per cent, nor the manganese content 1.00 per cent (by ladle analysis).

2.06 *Copper Bearing Steel.* Copper bearing steel with content of about 0.20 per cent copper may be used when specified. In other particulars the steel shall conform to specifications enumerated above.

2.07 *Rivets.* Rivets shall comply with the latest revision of A.S.T.M. Specifications A 31 or A 141. If cold driving is done, properly annealed rivets conforming to A.S.T.M. Specifications A 31 should be used.

2.08 *Pins.* Pins shall comply with A.S.T.M. Specifications A 108 Grade 5, latest revision.

2.09 *Castings.* Castings for riveted construction shall conform to the latest revision of A.S.T.M. Specifications A 27, Grade A-3.

Castings for welded construction shall conform to the latest revision of A.S.T.M. Specifications A 215, Grade A-3-W (carbon content not to exceed 0.25 per cent by ladle analysis).

2.10 *Forgings.* Forgings shall conform to the latest revision of one of the following A.S.T.M. Specifications: (For welded construction in no case shall the carbon content exceed 0.30 per cent by ladle analysis.)

(*a*) Plate Forgings—A 78 and A 89 (for welded construction specify for fusion welding).

(*b*) Forgings, other than plate,—A 18, Class B.

(*c*) Forged and Rolled Pipe Flanges,—A 181, Class I.

2.11 *Filler Metal.* The specifications cited below shall be followed for filler metal:

(*a*) Electrodes shall conform to the A.W.S.—A.S.T.M. "Tentative Specifications for Iron and Steel Arc Welding Electrodes," serial designation A 233-40T, using the following classifications: E 6010, E 6011, E 6012, E 6013, E 6020 and E 6030.

The classification number selected must be suitable for the electric current characteristics and also for the position of welding.

(*b*) Gas welding rods shall conform to the A.W.S.—A.S.T.M. "Tentative Specifications for Iron and Steel Gas Welding Rods," using the G-60 classification.

Section 3—General Design

3.01 *Type of Joints.* Unless specifically restricted by the purchaser, joints and connections in structures built under these specifi-

cations may be either riveted or welded or parts may be riveted or welded at the option of the contractor.

Lap welded joints between plates in contact with tank contents except flat bottom plates supported directly on grade or foundation shall be welded continuously on both edges.

3.02 *Design Loads.* The following loads shall be considered in the design of tank structures:

(*a*) *Dead load* shall be the estimated weight of all permanent construction and fittings. The unit weight of steel shall be considered 490 lb. per cu.ft. and of concrete 144 lb. per cu.ft.

(*b*) *Live load* shall be the weight of all of the liquid when the tank is filled to overflowing. Unit weight of water shall be considered as 62.4 lb. per cu.ft. The weight of any water, supported directly on foundations, shall not be considered as a live load on the superstructure.

(*c*) *Snow load* shall be 25 lb. per sq.ft. of the horizontal projection of the tank for surfaces having a slope of less than 30° with the horizontal. For greater roof slope snow loads shall be neglected.

(*d*) *Wind load* or pressure shall be assumed to be 30 lb. on vertical plane surfaces, 18 lb. on projected areas of cylindrical surfaces and 15 lb. per sq.ft. on projected areas of conical and double curved plate surfaces. (It may be desirable to increase the wind loads above by as much as 50 per cent in hurricane areas.)

For columns and struts of structural shapes the equivalent flat vertical surface area shall be calculated.

In calculating the wind load on elevated tank structures the entire wind load on the tank, roof and bottom and the proper proportion of the wind load from the riser pipe and tower shall be assumed to act on the tank at the center of gravity of such loads.

(*e*) *Earthquake Load:* If any provision is to be made in the design for earthquake resistance the purchaser shall so specify. *Note:* Present practice is to assume a horizontal force of from 5 to 20 per cent of the total weight of the water in the tank and riser acting at the center of mass of the water, the percentage used depending upon the proximity of an earthquake-producing fault and the hazard to surrounding property in the event of an earthquake failure or upon local regulations.

(*f*) *Balcony and Ladder Load:* A vertical load of 1,000 lb. shall be assumed to be applied to any point on the balcony floor, if any, and on each platform; 500 lb. at any point on the tank roof; 350 lb. on each section of ladder; and all of the structural parts and connections shall be properly proportioned to withstand such loads.

3.03 *Unit Stresses.* With the exceptions specifically provided for elsewhere in these specifications, all steel members shall be so designed and proportioned that, during the application of any of the loads previously specified, or any combination of them, the maximum stress shall not exceed the following:

	Maximum Fiber Stress Pounds Per Square Inch
(*a*) *Tension:*	
Structural steel	15,000
Rivets, on area based on nominal diameter	11,250
Bolts, on area based on diameter at root of threads	11,250
Cast steel	11,250
(*b*) *Compression:*	
Structural steel and weld metal	15,000
Columns and struts	See Section 3.05
Plate girder stiffeners	15,000
Webs of rolled sections at toe of fillet	18,000
Cast steel	15,000
(*c*) *Bending:*	
Tension on extreme fibers, except column base plates	15,000
Column base plates	20,000
Compression on extreme fibers of rolled sections, and built-up girders and built-up members, for values of $\frac{l}{b}$ not greater than 40, where l is laterally unsupported length of the member and b is the width of the compression flange	$\dfrac{16,850}{1 + \dfrac{l^2}{1,800b^2}}$ with a maximum of 15,000
Pins, extreme fiber	22,500
Cast steel	11,250
(*d*) *Shearing:*	
Rivets, pins and turned bolts in reamed or drilled holes	11,250
Unfinished bolts	7,500
Webs of beams and plate girders, gross section	9,750
Cast steel	7,325

(*e*) *Bearing:*

	Double Shear	Single Shear
Rivets, and turned bolts in reamed or drilled holes	30,000	24,000
Unfinished bolts	18,750	15,000
Pins	24,000	24,000
Contact area of milled surfaces	22,500	
Contact area of fitted stiffeners	20,250	
Expansion rollers and rockers (pounds per lineal inch) where d is the diameter of roller or rocker in inches		$600d$
Concrete (see Section A-22)		
2,000 lb. Concrete		500
2,500 lb. Concrete		625
3,000 lb. Concrete		750

Unit stress values wherever stated in these specifications shall be reduced by multiplying them by the applicable joint efficiencies.

3.04 *Combined Stresses.*

(*a*) *Axial and Bending:* Members subjected to both axial and bending stresses shall be so proportioned that the quantity $\frac{fa}{Fa} + \frac{fb}{Fb}$ shall not exceed unity, in which:

Fa is the axial unit stress that would be permitted by this specification if axial stress only existed.

Fb is the bending unit stress that would be permitted by this specification if bending stress only existed.

fa is the axial unit stress (actual) = axial stress divided by area of member.

fb is the bending unit stress (actual) = bending moment divided by section modulus of member.

(*b*) *Rivets:* Rivets subject to shearing and tensile forces shall be so proportioned that the combined unit stress will not exceed the allowable unit stress for rivets in tension only.

(*c*) *Wind and Other Forces:* Members subject to stresses produced by a combination of wind and/or earthquake and other loads may be proportioned for unit stresses 25 per cent greater than those specified in Sections 3.03 and 3.05, provided the section thus required is not less than that required for the combination of dead load and live load.

Members subject only to stresses produced by wind forces and/or earthquake forces may be proportioned for unit stresses 25 per cent greater than those specified in Section 3.03.

3.05 *Column and Strut Formulas.* The following formulas shall be applied:

(*a*) *Structural Sections:* The maximum permissible unit stress for structural columns or struts shall be determined from the following formula:

$$\frac{P}{A} = \left[\frac{18000}{1 + \frac{L^2}{18000r^2}}\right]$$

$\frac{P}{A}$ in no case to exceed 15,000 lb. per sq.in.

(*b*) *Tubular Sections:* The maximum permissible stress for tubular columns and struts shall be determined by the following formula.

$$\frac{P}{A} = \left[\frac{18000}{1 + \frac{L^2}{18000r^2}}\right]\left(\frac{2}{3}\right)\left(100\frac{t}{R}\right)\left[2 - \left(\frac{2}{3}\right)\left(100\frac{t}{R}\right)\right]$$

$\frac{P}{A}$ shall, in no case, exceed 15,000 lb. per sq.in.

In the above formula the term $\left[\frac{18000}{1 + \frac{L^2}{18000r^2}}\right]$ shall not exceed 15,000 lb. per sq.in.

In the above formula the term $\left(\frac{2}{3}\right)\left(100\frac{t}{R}\right)\left[2 - \left(\frac{2}{3}\right)\left(100\frac{t}{R}\right)\right]$ shall be evaluated as unity (one) for values of $\frac{t}{R}$ equal to or exceeding 0.015.

In the foregoing formulas the symbols have the following meaning:

P = the total axial load in pounds.
A = the cross sectional area in square inches.
L = the effective length in inches.
r = the least radius of gyration in inches.
R = the radius of the tubular member to the exterior surface in inches.
t = the thickness of the tubular member in inches, not less than $\frac{1}{4}$ in.

3.06 *Slenderness Ratio.* The maximum permissible slenderness ratio $\left(\frac{L}{r}\right)$ for compression members carrying weight or pressure of tank contents shall be 120.

The maximum permissible slenderness ratio $\left(\frac{L}{r}\right)$ for compression members carrying loads from wind and earthquake only shall be 175.

The maximum permissible slenderness ratio $\left(\frac{L}{r}\right)$ for columns carrying roof loads only shall be 175.

3.07 *Roofs and Top Girders.* All tanks storing drinking water should have roofs. Tanks without roofs shall have a top girder or angle having a minimum section modulus as determined by the following formula:

$$S = \frac{HD^2}{10000}$$

In the above formula S is the minimum required section modulus in inches cubed of the top angle or girder, including a length of tank shell equal to twenty times its thickness; H is the height of the cylindrical portion of the tank shell in feet; and D is the diameter of the cylindrical portion of the tank shell in feet.

3.08 *Roof Supports.* Roof supports, if any, shall be designed in accordance with the current specifications of the American Institute of Steel Construction, except that the ratio $\frac{l}{b}$ of unbraced length to width of flange of rafters in contact with roof sheets shall not be restricted, as it is considered that the roof sheets will provide lateral supports for the rafters, and except that the maximum slenderness ratio $\frac{L}{r}$ for columns supporting roofs shall be 175.

Roof trusses shall be placed above the maximum water level in climates where ice may form.

Roof rafters shall preferably be placed above the maximum water level but it will be permissible for their lower ends, where they connect to the tank shell, to project below the water level.

3.09 *Foundation Bolts.* Foundation bolts may be either plain or deformed bars, either upset or not upset. They shall be proportioned for the maximum possible uplift, using the area at the base of the thread or the un-upset rod diameter whichever is smaller. Foundation bolts may extend into the pier to within 3 in. from the bottom of the pier, but not necessarily more than far enough to develop the maximum uplift, and shall terminate in a right angle bend, hook or washer plate. The threaded ends of foundation bolts shall extend 2 in. above the nominal level of the tops of the foundation bolt nuts

to provide for variations in the foundations. Lock nuts shall be provided or the threaded ends of anchor bolts shall be peened to prevent loosening of anchor nuts.

3.10 *Corrosion Allowance.* Careful consideration shall be given by the purchaser to the proper allowance for corrosion. This allowance will depend upon the corrosive nature of the stored water, the proximity of the tank to salt water or other causes of atmospheric corrosion and the care with which the paint or other protection will be maintained. The corrosion allowance specified by the purchaser is to be added to the thickness determined from the specified design units or to the minimum thicknesses specified in the following section.

3.11 *Minimum Thickness.* The minimum thickness for any part of the structure shall be $\frac{3}{16}$ in. for parts not in contact and $\frac{1}{4}$ in. for parts in contact with water contents. Round or square bars used for wind bracing or for miscellaneous purposes shall have a minimum diameter or width of $\frac{5}{8}$ in. Bars of other shapes, if used, shall have a total area at least equal to a $\frac{5}{8}$ in. round bar.

3.12 *Joints in Shell Plates.* In calculating the thicknesses of plates stressed by the weight or pressure of the tank contents, the pressure at the lower edge of each ring shall be assumed to act undiminished on the entire area of the ring. Joints subjected to stress from weight or pressure of tank contents in adjacent courses shall be staggered by at least one-fourth of the distance between such joints.

3.13 *Riveted Joint Design.* The riveted joint design shall comply with current good practice as regards the size of rivets, edge distance for calking and non-calking edges, the angle of bevel of calking edges and the minimum and maximum rivet pitches and back pitches.

The following represents good practice:

The minimum rivet pitch shall be not less than three times the nominal diameter of the rivet.

The maximum pitch along a calked edge, except for column connections, shall not exceed 2.5 times the thickness of the thinnest plate connected for single rivet joints or 3 times the thickness of the thinnest plate in joints having more than one row of rivets, plus, in each case, the diameter of the rivet hole in inches plus 1.5 in.

Maximum pitch along uncalked edges shall not exceed 30 times the thickness of the thinnest plate connected.

The distance between the center of the outer row of rivets and the edge of plate shall not be less than 1.5 times the diameter of the rivets. Where edges are beveled, the distance from the center of rivets to the toe of the bevel shall not exceed 1.75 times the diameter of the rivets.

Spacing between rows of rivets shall be as follows:

(*a*) For joints where one rivet in the inner row comes midway between two rivets in the outer row, the spacing between the rivets or back-pitch shall be $1.75d + 0.185\ (P - 4d)$, with a minimum of $1.75d$.

(*b*) For joints where two rivets in the inner row are placed between two rivets in the outer row, the back-pitch shall equal $2d + 0.14\ (P - 4d)$, with a minimum of $2d$.

P is the pitch, in inches, of the rivets in the outer row.

d is the diameter, in inches, of the rivet holes.

3.14 *Riveted Joint Efficiency.* The joint efficiency for riveted joints shall be calculated on the basis of the previously specified unit stresses.

For punched holes the shear and bearing of rivets shall be based on the nominal rivet diameter and the net tension or tearing shall be based on the nominal rivet diameter plus $\frac{1}{8}$ in.

For drilled or subpunched and reamed holes the shear, bearing and tension or tearing shall be based on the drilled or reamed diameter of the holes.

The joint shall be designed and the efficiency determined before adding the corrosion allowance.

3.15 *Design Stresses—Fillet Welds.* Stress in a fillet weld shall be considered as shear. The values shall be in pounds per square inch of cross-section, measured in the throat of the weld. The throat of a fillet weld shall be assumed as 0.707 times the length of the shorter leg of the fillet weld.

(*a*) *Transverse Shear:* The stress in fillet welds acting in transverse shear, for structural or other attachments, shall be computed as 65 per cent of the allowable working tensile stress of the plate or structural material joined.

(*b*) *Longitudinal Shear:* The stress in fillet welds acting in longitudinal shear, for structural or other attachments, shall be computed as 50 per cent of the allowable working tensile stress of the plate or structural material joined.

3.16 *Welded Joint Efficiencies.*

Type of Joint	*Efficiency Per Cent*
(a) Double-welded butt joint with complete penetration.	85 Tension; 100 Compression
(b) Double-welded butt joint with partial penetration and with the unwelded portion located substantially at the middle of the thinner plate.	$85\ \frac{Z}{T}$ Tension; $100\ \frac{Z}{T}$ Compression

Where Z is the total depth of penetration from the surfaces of the plate, (use the thinner plate if of different thicknesses);
T is the thickness of the plate, (use the thinner plate if of different thicknesses).

(c) Single-welded butt joint with suitable backing-up strip or equivalent means to insure complete penetration.	85 Tension; 100 Compression
(d) Double-welded lap joint with full-fillet weld on each edge of joint.	75 Tension or Compression
(e) Double-welded lap joint with full-fillet weld on one edge of joint and an intermittent full-fillet weld on the other edge of joint.	$75\frac{(1 + X)}{(2)}$ Tension or Compression

Where X is the percentage of full-fillet intermittent welding, expressed as a decimal.

(f) Lap joint with full-fillet welds, or smaller, on either or both edges of the joint, welds either continuous or intermittent.	$75\frac{(XW_1 + YW_2)}{2T}$ Tension or Compression

Where X and Y are the percentages of intermittent welds for welds W_1 and W_2 respectively, expressed as a decimal.

W_1 and W_2 are the sizes of the welds on each edge of the joint respectively.
(W_2 will be zero for a joint welded on one edge only.)
T is the thickness of plate, (use the thinner plate if of different thicknesses).

3.17 *Reinforcement Around Openings.* All openings over 4 in. in diameter in the shell or suspended bottom of the tank shall be reinforced. This reinforcement may be the flange of a fitting used or an additional ring of metal, or both flange and ring.

The amount of reinforcement for a riveted tank shell shall be computed as follows:

(*a*) In computing the necessary reinforcement of an opening in a tank shell, the net area of the reinforcement shall bear the same

ratio to the area of the metal removed from the shell as the strength of the vertical joint in the shell course bears to the strength of the solid plate.

(*b*) Sufficient rivets shall be used to transmit to the shell plate by shear the full strength of the reinforcing ring or flange, at a maximum unit shear of 11,250 lb. per sq.in.

The amount of reinforcement for a welded tank shell shall be computed as follows:

(*c*) The strength of the required reinforcement around an opening in a shell plate shall be based on the vertical cross-sectional area of the metal removed from the plate. This area shall be taken as the product of the vertical diameter of the hole cut in the shell plate and the thickness of the plate (100 per cent reinforcement). The strength through the net cross-sectional area of the reinforcement added, lying in a vertical plane (plane of reinforcement) coincident with the axis of the opening, shall at least equal that of the product referred to above.

(*d*) The aggregate strength of the welding attaching a fitting to the shell plate and/or any intervening reinforcing plate shall at least equal the proportion of the forces passing through the entire reinforcement that is computed to pass through the fitting considered.

(*e*) The aggregate strength of the welding attaching any intervening reinforcing plate to the shell plate shall at least equal the proportion of the forces passing through the entire reinforcement, that is computed to pass through the reinforcing plate considered.

(*f*) The strength of the attachment welding shall be based only on that part of the outer peripheral welding which lies on either side outside the area bounded by vertical lines drawn tangent to the shell opening plus all of the inner peripheral welding that is applied on either side of the plane of reinforcement. This shall be taken as the total shear resistance of the above mentioned attachment welding. The outer peripheral weld shall be made as large as possible and the inner peripheral weld large enough to carry the remainder of the total loading.

(*g*) In computing the net reinforcing area of a fitting, such as a boiler maker's flange, or a manhole flange having a neck, the material in the neck may be considered as part of the flange, for a height, measured from the surface of the shell plate or that of an intervening reinforcement plate, equal to four times the thickness of the material in the neck.

Section 4—Design of Standpipes and Reservoirs

4.01 *Standard Capacities.* The standard capacities for standpipes and reservoirs shall be those recommended by the Division of Simplified Practice of the Department of Commerce and shall be as follows.

United States Gallons	United States Gallons
50,000	500,000
60,000	750,000
75,000	1,000,000
100,000	1,500,000
150,000	2,000,000
200,000	2,500,000
250,000	3,000,000
300,000	4,000,000
400,000	

4.02 *Standard Shell Height for Standpipes.* The purchaser shall preferably specify one of the following standard shell heights for standpipes:

Shell Heights from 20 ft. to 50 ft. by even 2-foot intervals
Shell Heights from 50 ft. to 100 ft. by even 5-foot intervals
Shell Heights from 100 ft. to 200 ft. by even 10-foot intervals

The purchaser shall preferably specify the required shell height and the capacity, the exact diameter being determined by the contractor. As here used shell height means height to overflow.

4.03 *Standard Diameters for Reservoirs.* The purchaser shall preferably specify one of the following standard diameters for reservoirs in order to promote standardization of drawings:

Diameters from 20 ft. to 50 ft. by even 2-foot intervals.
Diameters from 50 ft. to 100 ft. by even 5-foot intervals.
Diameters from 100 ft. to 200 ft. by even 10-foot intervals.

For reservoirs the purchaser shall specify the diameter and capacity, the exact height being determined by the contractor.

Section 5—Design of Elevated Tanks

5.01 *Standard Capacities.* The standard capacities for elevated tanks shall preferably be those recommended by the Division of Simplified Practice of the Department of Commerce as follows.

United States Gallons	United States Gallons	United States Gallons
5,000	60,000	500,000
10,000	75,000	750,000
15,000	100,000	1,000,000
20,000	150,000	1,500,000
25,000	200,000	2,000,000
30,000	250,000	2,500,000
40,000	300,000	
50,000	400,000	

5.02 *Standard Heights for Elevated Tanks.* The height of elevated tank structures shall be measured from the underside of the bases of the steel columns to the lower capacity level. The purchaser shall preferably specify one of the following standard heights:

Heights from 20 ft. to 50 ft. by even 2-foot intervals.
Heights from 50 ft. to 100 ft. by even 5-foot intervals.
Heights from 100 ft. to 200 ft. by even 10-foot intervals.

5.03 *Standard Ranges of Head.* By range of head is meant the vertical distance between the lower capacity level and the overflow between which the required capacity is contained. Where range of head is not material the purchaser shall preferably leave the determination of the range of head to the contractor.

If a special "low" range of head is required the purchaser shall preferably specify ranges of head by even 5-foot intervals.

5.04 *Columns and Struts.* The column base, whether riveted or welded, shall have sufficient area to distribute the column load over the concrete foundations without exceeding the specified bearing stress on the foundation and the connection shall provide for the maximum uplift, if the anchors are connected to the base plates and not to the column shaft.

5.05 *Column Splices.* Column splices shall be designed to withstand the maximum possible uplift.

If column splices are riveted, the flanges only of rolled column

sections need be spliced. Rolled channels, if used in columns, shall have both the flanges and webs spliced. For welded column splices the joints may either be butt welded or splice plates may be welded to both sections being joined, the welds developing sufficient strength to resist the maximum uplift or at least 25 per cent of the maximum compression, whichever is greater.

5.06 *Bottom Struts.* Bottom struts of steel or reinforced concrete shall be provided where necessary to distribute the horizontal reactions at the bases of the columns.

5.07 *Tension Members Carrying Wind and/or Earthquake Loads.* Such members shall be designed to resist the wind load and the earthquake load, if the latter is specified. If the projected centers of gravity of tension members do not meet the projected center of gravity of strut members at the center of gravity of the columns, proper allowance shall be made for the eccentricity.

Diagonal tension members shall be pre-stressed sufficiently to be taut when the tank is full. Such pre-stressing shall not be given consideration in the design of the members.

5.08 *Horizontal Girders.* For elevated tanks with inclined or battered columns connecting to the tank shell a horizontal girder shall be provided to resist the horizontal component of the column loads.

This girder shall be proportioned to withstand safely as a ring girder the horizontal inward component of the load on the top columns.

If the centers of gravity of the horizontal girder, the top section columns and the tank shell do not meet in a point, provision shall be made in the design of each of them for stresses resulting from any eccentricity.

5.09 *Balcony Railing.* If a horizontal girder is used as a balcony it shall be at least 24 in. in width and shall be provided with a railing at least 36 in. in height.

5.10 *Tank Plates.* Plates for elevated tank bottoms, shells and roofs may be any desired shape. Tank plates shall be designed on the basis of the following maximum fiber stresses which shall be reduced for the joint efficiencies as specified elsewhere in these specifications.

Plate surfaces susceptible to complete stress analysis shall be designed on the basis of a maximum fiber stress of 15,000 lb. per sq.in. Such plate surfaces include those not stressed by the concentrated reactions of supporting members or riser pipes.

Plate surfaces not susceptible to complete stress analysis shall also

be designed on the basis of the maximum fiber stress of 15,000 lb. per sq.in. after making reasonable allowances for such loads and stresses as cannot be accurately determined. The maximum fiber stress shall in no case exceed 11,000 lb. per sq.in. when calculated assuming that the concentrated reactions of supporting members are uniformly distributed between such reactions.

For example, consider an elevated tank having a vertical cylindrical shell supported by four columns attached to the shell and having a suspended ellipsoidal bottom with a central riser pipe and a cone roof uniformly supported by the tank shell. Under the meaning of these specifications, the stresses in the cylindrical shell and the ellipsoidal bottom cannot be determined, while those in the roof can be completely determined. The shell and bottom shall be designed on the basis of 15,000 lb. per sq.in. maximum fiber stress reduced for the joint efficiency used making allowances for the following:

(*a*) The hoop stresses caused by the weight or pressure of the tank contents, assuming that the cylindrical tank shell is uniformly supported on its entire lower circumference.

(*b*) The stresses in the cylindrical shell and ellipsoidal bottom, considering them acting together as a circular girder supported by the column reactions and subjected to torsion because of the portions projecting outward and inward from the chords connecting the columns.

(*c*) The horizontal inward component of the pull from the tank bottom (in the case of conical or segmental bottoms) causing compression in the tank shell.

(*d*) Stresses from any other causes.

After the cylindrical shell and bottom have been designed on the above basis, they shall be redesigned assuming that the cylindrical tank shell is uniformly supported on its entire lower circumference, and for this assumption the thicknesses of shell and bottom shall be increased, if necessary, so that the maximum calculated fiber stress shall not exceed 11,000 lb. per sq.in. reduced by the joint efficiency.

In designing bottoms of double curvature, consideration shall be given to the possibility of governing compressive stresses.

It is recognized that no specifications for the design of elevated tanks can be sufficiently specific and complete to eliminate the necessity of judgment on the part of the designer. It is also recognized that strain gage surveys are a proper source of design information.

5.11 *Steel Riser Pipe.* The steel riser pipe shall be designed to withstand stress caused by the weight or the pressure of the tank and riser contents and also the load imposed upon the top of the riser pipe by the tank bottom or by members supporting the tank bottom. If the design of the riser plates is controlled by hoop pressure, 0.3 of the compression in the vertical direction shall be added to the full calculated tension in the horizontal direction in determining the thickness. If controlled by column stresses the allowable column compressive stress shall not exceed the column stress calculated in accordance with Section 3.05, minus 0.3 of the calculated hoop stress.

The thickness of the bottom ring of the riser shall be sufficient so that the specified unit stresses shall not be exceeded when combined with bending or other stresses around manhole or other openings.

Section 6—Accessories for Standpipes and Reservoirs

6.01 *Shell Manhole.* A circular manhole 24 in. in diameter or an elliptical manhole 18 in. x 22 in. minimum size, with cover hinged to shell shall be furnished in the first ring of the tank shell at a location to be designated by the purchaser.

6.02 *Pipe Connection.* The pipe connection shall consist of a fitting of the size specified by the purchaser, attached to the tank bottom at a point designated by the purchaser, into which the connecting pipe may be calked. Unless otherwise specified by the purchaser, the contractor shall furnish the fitting and make the connection to the piping furnished and installed by the purchaser. The top of the fitting shall be flush with the tank floor and provided with a removable silt stop 6 in. high.

6.03 *Overflow.* The tank shall be equipped with an overflow of the type and size specified by the purchaser. A stub overflow is recommended in cold climates. If a stub overflow is specified it shall project at least 12 in. beyond the tank shell. If an overflow to ground is specified it shall be brought down the outside of the tank shell supported at proper intervals with suitable brackets. It shall terminate at the top in a weir box, the weir and connection to the tank to have approximately the same capacity as the overflow pipe specified by the purchaser allowing for full suction head. The top angle shall not be cut nor partially removed. The overflow pipe shall terminate at the bottom with a base ell. Unless otherwise specified by the purchaser, the overflow pipe shall be black steel pipe, with screwed connections if under 4 in. in diameter or flanged connections if 4 in. in diameter or over.

6.04 *Outside Tank Ladder.* The contractor shall furnish a tank

ladder on the outside of the tank shell beginning 8 ft. above the level of the tank bottom and at a location to be designated by the purchaser, preferably near the manhole. The sides shall be not less than 2 in. x $\frac{5}{16}$ in. and the rungs not less than $\frac{5}{8}$ in. round or square bars. If a safety cage for this ladder is required by local laws or regulations the purchaser shall so specify.

6.05 *Roof Ladder.* In the case of standpipes with roofs, the contractor shall furnish a roof ladder attached to the roof finial with a swivel connection, the ladder to be equipped with rollers so that it will rotate around the standpipe roof.

6.06 *Roof Hatch.* For standpipes or reservoirs with roofs, the contractor shall furnish a roof door or hatch, which shall be placed near the outside tank ladder and which shall be provided with hinges and a hasp for locking. The hatch opening shall have a curb at least 4 in. high and the cover shall overlap same at least 2 in.

6.07 *Vent.* In case the tank roof is of tight construction, a suitable vent shall be furnished above the maximum water level which shall have a capacity to pass air so that at the maximum possible rate of the water either entering or leaving the tank, dangerous pressures will not be developed. The overflow pipe shall not be considered to be a tank vent. The tank vent may, however, be combined with the roof finial if desired. The vent shall be so designed and constructed as to prevent the ingress of birds or animals.

6.08 *Additional Accessories.* Any additional accessories required to be furnished shall be specified by the purchaser.

Section 7—Accessories for Elevated Tanks

7.01 *Tower Ladder.* A tower ladder with sides not less than 2 in. x $\frac{5}{16}$ in. and rungs not less than $\frac{5}{8}$ in. round or square shall be furnished extending from a point 8 ft. above the ground up to and connecting with either the horizontal balcony girder or the roof ladder, if no balcony is used. The ladder may be vertical but shall not in any place have a backward slope.

7.02 *Outside Tank Ladder.* In all cases, a ladder shall be provided on the outside of the tank shell connecting either with the balcony or with the tower ladder if no balcony is included. The outside tank ladder shall have sides not less than 2 in. x $\frac{5}{16}$ in. and rungs not less than $\frac{5}{8}$ in. round or square.

7.03 *Roof Hatch.* In all cases, there shall be provided a door or hatch immediately above the high water level, the hatch to be at least 24 in. square and to be provided with suitable hinges and a hasp to permit locking. Hatch opening shall have a curb at least 4 in. high and the cover shall overlap same at least 2 in.

7.04 *Roof Finial.* The roof shall be provided with a suitable finial.

7.05 *Roof Ladder.* Where practicable, there shall be furnished an outside roof ladder designed to rotate around the roof. If desired the roof ladder may be swiveled about the tank finial.

7.06 *Vent.* In case the tank roof is of tight construction, a suitable vent shall be furnished above the maximum water level which shall have a capacity to pass air so that at the maximum possible rate of the water either entering or leaving the tank, dangerous pressures will not be developed. The overflow pipe shall not be considered to be a tank vent. The tank vent may, however, be combined with the roof finial if desired. The vent shall be so designed and constructed as to prevent the ingress of birds or animals.

7.07 *Steel Riser Pipe.* In localities where freezing temperatures do not occur, the purchaser may specify a small steel riser pipe. In other locations and unless a small pipe is specified a steel riser pipe not less than 36 in. in inside diameter shall be furnished. In cases where the riser pipe supports a considerable load the riser diameter and thickness shall preferably be determined by the contractor.

In all cases where steel risers are used, there shall be furnished a manhole in the riser shell about three feet from the base of the riser, such manhole to be not less than 12 in. x 16 in. in size, the opening to be reinforced or the riser plate so designed that all stresses around the opening are provided for.

The above specified minimum riser diameter of 36 in. shall be increased in cold climates unless the riser is heated to prevent freezing. The proper diameter will depend upon the amount the tank is used and the temperature of the water supplied. In extreme cold climates a minimum diameter of 72 in. is recommended.

A safety grating shall be provided in the top of the riser pipe with no opening larger than 6 in. in width, except that a door at least 12 in. x 18 in. shall be provided.

7.08 *Pipe Connection.* The pipe connection into which the connecting pipe may be calked, shall consist of a fitting of the size specified by the purchaser, and shall be attached to the riser bottom at a point designated by the purchaser. Unless otherwise specified by the purchaser, the contractor shall furnish the fitting and make the connection to the piping furnished and installed by the purchaser. The top of the fitting shall be flush with the riser floor and provided with a removable silt stop 6 in. high.

7.09 *Overflow.* The tank shall be equipped with an overflow of the type and size specified by the purchaser. A stub overflow is recommended in cold climates. If a stub overflow is specified, it

shall project at least 12 in. beyond the tank shell. If an overflow to ground is specified, it shall be brought down the outside of the tank shell, supported at proper intervals with suitable brackets. It shall terminate at the top in a weir box, the weir and connection to the tank to have approximately the same capacity as the overflow pipe specified by the purchaser, allowing for full suction head. The top angle shall not be cut or partially removed. The overflow pipe shall terminate at the bottom with a base ell. Unless otherwise specified by the purchaser, the overflow pipe shall be black steel pipe, with screwed connections if under 4 in. in diameter or flanged connections if 4 in. or over in diameter.

7.10 *Additional Accessories.* Any additional accessories required to be furnished shall be specified by the purchaser.

Section 8—Welding

8.01 *Definitions and Symbols.*

(*a*) Welding terms employed in these rules shall be interpreted according to the definitions given in the latest edition of "Definitions of Welding Terms and Master Chart of Welding Processes," issued by the American Welding Society.

(*b*) Symbols used on construction drawings shall conform to those shown in the latest edition of "Welding Symbols and Instructions for Their Use," issued by the American Welding Society.

8.02 *Qualification of Welding Procedure and Testing of Welding Operators.* Welding procedure shall be qualified and welding operators shall be tested according to the latest rules of the American Welding Society for Standard Qualification Procedure, using the following values and modifying conditions:

(*a*) *Butt Welds for Primary Stress:*

(Joints subject to primary stress due to weight or pressure of tank contents.)

(1) Tension Test—Reduced-Section Tension Test—not less than 95 per cent of the minimum of the specified A.S.T.M. tensile range of the base material.

(2) Free Bend Test—minimum elongation 20 per cent.

(3) Root, Face and Side Bend Test—Any specimen in which a crack is present after the bending exceeding $\frac{1}{8}$ in. measured in any direction shall be considered as having failed. Cracks occurring on the corners of the specimen during testing shall not be considered.

(*b*) *Butt Welds for Secondary Stress:*

(Joints subject to secondary stress, those not directly affected by weight or pressure of tank contents.)

(1) Tension Test—Butt joints which do not require full penetration shall have the procedure qualified by the Reduced-Section Tension Test only. This test shall not be used for the qualification of welding operators. (See Section 8.04.)

(2) The Reduced-Section Tensile Test shall give values not less than 110 per cent of the specified strength of the joint.

(c) *Fillet Welds:*

(1) Transverse Shear Test—shall have a minimum value in pounds per square inch equal to $\frac{7}{8}$ of the minimum of the specified A.S.T.M. tensile range of the base material.

(2) Longitudinal Shear Test—shall have a minimum value in pounds per square inch equal to $\frac{2}{3}$ of the minimum of the specified A.S.T.M. tensile range of the base material.

(3) Fillet Weld Soundness Test—Any specimen in which a crack is present after the bending exceeding $\frac{1}{8}$ in. measured in any direction shall be rejected. Cracks occurring on the corners of the specimens during testing shall not be considered.

(4) Double-fillet-welded lap joints using various size welds, such as used to resist secondary stress, may be qualified with shear tests only, using the unit shear stress values given in paragraphs (1) and (2). Such tests shall not be used for the qualification of welding operators.

(d) *General:*

(1) An operator who has been qualified for welding on plate will be permitted to perform the incidental pipe welding required in the fabrication and attachment of tank appurtenances.

(2) The contractor shall submit qualification test reports for the procedure he intends to use—including all types of welded joints, welded in all positions to be used in the construction. These test reports shall bear proper witness certification of a reputable testing laboratory or inspection agency, stating that such tests were performed in accordance with the above mentioned requirements. The tests need not be repeated provided the procedure is not modified.

(3) All welding operators assigned to weld on tanks constructed under these rules shall have been tested under the

above mentioned American Welding Society "Standard Qualification Procedure." The contractor shall certify that such tests were performed in accordance with the above mentioned requirements.

(4) The operator qualification tests herein specified shall be considered as remaining in effect indefinitely unless (1) the welding operator has not been continuously employed by the contractor on similar work, using similar welding procedure, for a period of three months or more since making qualification tests; or unless (2) there is some specific reason to question an operator's ability. In case (1) above, the requalification test need be made only in the $\frac{3}{8}$ in. thickness.

(5) The contractor shall maintain a record of each welding operator employed by him, showing the dates and results of qualification tests and the work-identification mark assigned to him.

8.03 *Butt-Welded Joints Subject to Primary Stress from Weight or Pressure of Tank Contents* (e.g., vertical joints of cylindrical tank shells; all joints below the point of support in suspended bottoms of elevated tanks).

These joints may be double welded to insure complete penetration; or they may be single welded with suitable backing-up strip or equivalent means to insure complete penetration.

8.04 *Butt-Welded Joints Subject to Secondary Stress* (e.g., horizontal joints of cylindrical tank shells). These joints shall be designed to develop an efficiency of at least 57 per cent, based on the thickness of the thinner plate joined. They shall be double welded and may have partial penetration (at least $\frac{2}{3}$ penetration required) with the unwelded portion located substantially at the middle of the thinner plate. (This joint shall be at least equivalent to $\frac{2}{3}$ that of a double welded butt joint with full penetration.)

8.05 *Lap Welded Joints Subject to Primary Stress from Weight or Pressure of Tank Contents* (e.g., vertical joints of cylindrical tank shells; all joints below the point of support in suspended bottoms of elevated tanks).

These joints shall have continuous full-fillet welds on both sides of the joint. (Maximum thickness permitted for this type of joint is $\frac{1}{2}$ in.)

8.06 *Lap Welded Joints Subject to Secondary Stress* (e.g., horizontal joints of cylindrical tank shells).

These joints shall be welded on both sides with continuous fillet welds. They shall be designed to develop an efficiency of at least

50 per cent, based upon the thickness of the thinner plate joined. (This joint shall be at least equivalent to $\frac{2}{3}$ that of a double full-fillet lap joint.) (Maximum thickness permitted for this type of joint — $\frac{5}{8}$ in.)

8.07 *Flat Tank Bottoms Resting Directly on Grade or Foundation.* Bottoms shall be built to one of two alternative methods of construction:

(*a*) Lap-welded construction—Bottom plates need be welded on the top side only with continuous full-fillet welds on all seams.

Marginal sketch plates under the bottom ring of cylindrical shells, in the case of lap welded construction, shall have the outer end of the joint fitted and butt or lap welded to form a smooth bearing under the shell. (Maximum thickness for lap welded bottoms shall be $\frac{1}{2}$ in.)

(*b*) Butt-welded construction—These joints shall be single welded from the top side, using suitable backing-up strip or equivalent means to insure complete penetration.

8.08 *Shell to Bottom Joint* (applies to vertical cylindrical shells with flat bottoms). The attachment between the bottom edge of the lowest course shell plate and the bottom sketch plates shall be by a continuous fillet weld laid on both sides of the shell plate. The size of each weld shall be equal to the thickness of the sketch plate or the thickness of the shell plate, whichever is smaller, with a maximum size of $\frac{1}{2}$ in. The sketch plates shall extend outside the tank shell a distance of at least 1 in. beyond the toe of the weld.

8.09 *Roof Plates.*

(*a*) Roofs not subject to hydrostatic pressure from tank contents—such roof plates need be welded on the top side only with continuous full-fillet welds on all seams.

(*b*) Roofs subject to hydrostatic pressure from tank contents—such roof plate joints shall be designed to conform to the efficiency values given in Section 3.16.

8.10 *Maximum Thickness of Material to Be Welded.*

(*a*) Maximum thickness permitted for lap welded joints subject to primary stress from weight or pressure of tank contents—$\frac{1}{2}$ in. (e.g., vertical joints of cylindrical tank shells; all joints below the points of support in elevated tanks).

(*b*) Maximum thickness permitted for lap welded joints, subject to secondary stress—$\frac{5}{8}$ in. (e.g., horizontal joints of cylindrical tank shells).

(*c*) Maximum thickness for lap welded flat tank bottoms resting directly on grade or foundation—$\frac{1}{2}$ in.

(*d*) Maximum thickness of metal permitted to be field welded—$1\frac{1}{2}$ in.

8.11 *Minimum Laps for Lap Welded Joints.*

(*a*) Joints subject to primary stress from weight or pressure of tank contents—5 *T* (e.g., vertical joints of cylindrical tank shells; all joints below the point of support in suspended bottoms).

(*b*) Joints subject to secondary stress—3 *T* with a minimum of 1 in. (e.g., horizontal joints of cylindrical tank shells, flat tank bottoms and roofs not subject to hydrostatic pressure from tank contents).

Where, *T* is the thickness of material welded, (use the thinner plate if of different thicknesses).

8.12 *Intermittent Welding.*

(*a*) The length of any segment of intermittent weld shall be not less than four (4) times the weld size with a minimum of $1\frac{1}{2}$ in.

(*b*) Intermittent welding shall not be used on tank shell plating.

(*c*) Intermittent butt welds shall not be used.

(*d*) All seams of intermittent welding shall have continuous welds at each end for a distance of at least 6 in.

8.13 *Minimum Size Fillet and Seal Welds.*

(*a*) Plates $\frac{3}{16}$ in. and less in thickness shall have full fillet welds. Plates over $\frac{3}{16}$ in. thick shall have welds of size not less than $\frac{1}{3}$ the thickness of the thinner plate at the joint, with a minimum of $\frac{3}{16}$ in.

(*b*) Sealing, when desired, shall not be accomplished by running a bead of minimum dimension along the intervals between intermittent strength welds, but by applying a continuous weld, combining the functions of sealing and strength and changing section only gradually and as changes in the required strength may necessitate.

8.14 *Minimum Length of Welds.*

(*a*) The minimum length of any weld shall be four (4) times the size, with a minimum of $1\frac{1}{2}$ in., or else the size of the weld shall be considered not to exceed one-fourth its length.

(*b*) The effective length of a fillet weld shall not include the length of tapered ends. A deduction of at least $\frac{1}{4}$ in. shall be made from the overall length as an allowance for the tapered ends.

Section 9—Shop Fabrication

9.01 *Workmanship.* All workmanship done on tanks being built under these specifications shall be first class in every respect.

9.02 *Straightening.* Any required straightening of material shall be done by methods which will not injure the steel. Straightening

by hammering will not be permitted but shall be done cold by rolling or pressing.

9.03 *Laying Out.* Laying out shall be done by experienced workmen only. Rivet holes shall be accurately spaced so that they come opposite each other in adjoining parts. If, upon assembling, holes do not match within $\frac{1}{8}$ in. for punched holes or $\frac{1}{16}$ in. for drilled or reamed holes, the inspector may order the contractor to ream the holes to a larger size and to use larger rivets, or, if the error is too great to permit this, the plate or plates containing such unfair holes may be rejected at the option of the inspector.

9.04 *Rivet Holes.* Rivet holes in material $\frac{1}{2}$ in. thick and under may be punched or drilled full size.

Rivet holes in material over $\frac{1}{2}$ in. to and including $\frac{3}{4}$ in. thick shall be either drilled from the solid or punched $\frac{1}{16}$ in. smaller in diameter than the nominal diameter of the rivet and then reamed to size.

Rivet holes in material over $\frac{3}{4}$ in. thick shall be drilled.

The final diameter of rivet holes shall be not more than $\frac{1}{16}$ in. larger than the rivets.

9.05 *Plate Edges—Riveted Work.* Edges of plates which are to be calked shall be beveled either by shearing, machining, or cutting with a machine operated gas torch, except that plates over $\frac{5}{8}$ in. thick shall not be sheared.

Plates $\frac{1}{2}$ in. and less in thickness may be beveled to approximately a 60- to 70-degree angle. Plates over $\frac{1}{2}$ in. in thickness may be beveled to an angle of approximately 75 to 80 degrees.

9.06 *Finish of Plate Edges—Welded Work.*

(*a*) The welding edges of plates may be universal mill edges or they may be prepared by shearing, machining, chipping, or by mechanically guided gas-cutting, except that irregular edges may be prepared by manually guided gas-cutting.

(*b*) When edges of plates are gas cut, the resulting surface must be uniform and smooth and must be freed of slag accumulations before welding. All cutting shall follow closely the lines prescribed.

(*c*) Shearing of material for butt-welded joints shall be limited to a thickness of $\frac{1}{2}$ in., or less. Material for all permitted thicknesses of lap-welded joints may be sheared.

9.07 *Scarfing for Riveted Construction.* Scarfing may be done either by heating the corners to be scarfed to a cherry-red color, then forging while hot, or the scarf may by forging in the cold either by hydraulic or mechanical pressing.

9.08 *Rolling.* Plates shall be cold rolled to the proper curvature in accordance with the following table.

Plate Thickness	*Minimum Diameter for Plates Not Rolled*
Plates less than $\frac{3}{8}$ in.	30 ft.
$\frac{3}{8}$ in. to less than $\frac{1}{2}$ in.	60 ft.
$\frac{1}{2}$ in. to less than $\frac{5}{8}$ in.	120 ft.
$\frac{5}{8}$ in. and heavier	Must be rolled for all diameters

All butt straps are to be formed to the proper curvature.

9.09 *Double Curved Plates.* Plates which are curved in two directions may be pressed either cold or hot or may be dished with a "mortar and pestle" die by repeated applications.

9.10 *Milling Columns.* The ends of columns shall be milled to provide a satisfactory bearing unless the design contemplates sufficient riveting or welding to resist the total calculated loads.

9.11 *Shop Assembly.* Double-curved tank bottoms, shells, and roofs, shall be assembled in the shop, if necessary, to insure their fitting properly in the field.

9.12 *Shipping.* All materials shall be loaded on cars, unloaded, transported to the site and stored in such manner as to prevent damage.

Section 10—Erection

10.01 *General.* The contractor shall furnish all labor, liability and compensation insurance, tools, falsework, scaffolding and other equipment necessary and erect the tank complete ready for use. He shall furnish to the purchaser certificates of insurance coverage.

10.02 *Riveted Tanks.* Plates shall be carefully and accurately laid up and shall then be firmly drawn together with machine bolts or wedge bolts before riveting is started.

No paint or foreign materials shall be used between surfaces in contact.

Rivets under $\frac{5}{8}$ in. in diameter may be driven hot or cold. All rivets $\frac{5}{8}$ in. in diameter and larger shall be driven hot.

Hot driven rivets shall be driven by pneumatic or hydraulic tools whenever possible.

Rivets shall be driven with either cone, steeple or button shaped snaps, except that cold rivets may be driven with mushroom heads.

Heads shall be as nearly as possible concentric with the rivet body. All rivets shall be driven from the side of the plate which calks.

Any burned, loose or defective rivets shall be cut out and redriven.

Riveted seams shall be made tight by calking. The portion of the

roof, if any, containing water shall be calked, the remainder of the roof need not be calked. Calking tools and methods used shall be such that the underlying sheet is not damaged and the edge of the top sheet is not turned under. The work must be done by experienced men only.

The opening between ends of plates and outside butt straps shall be stopped by means of a wedge or by welding.

10.03 *Welded Tanks.* If painting is required on underside of flat tank bottom, the bottom plates shall be spread out and painted. After the paint has dried the plates shall be moved to their proper location on the foundations and turned over in place with as little subsequent moving as possible in order not to damage the paint film.

The bottom plates shall be assembled and welded together following a procedure which will result in a minimum of distortion from weld shrinkage.

All shell, bottom and roof plates subjected to stress by the weight or pressure of the contained liquid shall be assembled and welded in such a manner that the proper curvature of the plates in both directions is maintained.

Extra holes in the plates for erection purposes may be used provided the holes are later filled with rivets or weld metal in compliance with Section 11.09.

Any clips, jigs or lugs welded to the shell plates for erection purposes shall be removed without damaging the plates and any portion of weld beads remaining shall be chipped or ground smooth.

10.04. *Welds—General.* All welds in the tank and structural attachments shall be made in a manner to insure complete fusion with the base metal, within the limits specified for each joint, and in strict accordance with the qualified procedure.

10.05. *Preparation of Welding Surfaces.* Surfaces to be welded shall be free from loose scale, slag, heavy rust, grease, paint and any other foreign material excepting tightly adherent mill scale. A light film of linseed oil or spatter film compound may be disregarded. Joint surfaces shall be smooth, uniform, and free from fins, tears, and other defects which adversely affect proper welding. A fine film of rust, adhering after wire brushing on cut or sheared edges that are to be welded, need not be removed.

10.06 *Weather Conditions.* Welding shall not be done when the temperature of the base metal is less than 0°F.; when surfaces are wet from rain, snow or ice; when rain or snow is falling on the surfaces to be welded; or during periods of high wind, unless the operator and the work are properly protected. At temperatures between

32°F. and 0°F., the surface within 3 in. of the point where the weld is to be started, shall be heated to a temperature warm to the hand before the welding is started.

10.07 *Cleaning Between Passes.* Each pass of weld metal on multi-pass welding shall be cleaned of slag and other loose deposits before applying the next pass.

10.08 *Tack Welds.* Erection tack-welds used in the assembly of joints subject to primary stress from weight or pressure of tank contents and those used for assembling the tank shell to bottoms are to be removed ahead of the continuous welding. Tack-welds used in the assembly of joints subject to secondary stress, such as those in flat bottoms, roofs and circumferential seams of cylindrical tank shells need not be removed, provided they are sound and the cover beads are thoroughly fused into the tack-welds.

10.09 *Peening.*

(*a*) Peening of weld layers or passes may be used to prevent undue distortion. Surface layers shall not be peened.

(*b*) Peening shall be performed with light blows from a power hammer using a blunt nosed tool.

10.10 *Weld Contour.*

(*a*) In all welds, the surface passes shall merge smoothly into each other.

(*b*) Undercutting of base metal in plate adjoining the weld is a defect and shall be repaired, except as permitted in Section 11.08 (f) and (g).

(*c*) All craters shall be filled to the full cross-section of the weld.

10.11 *Weld Reinforcement.* On butt joints, no part of the finished face in the area of fusion shall lie below the surface of the base plate adjoining. The weld reinforcement shall be a minimum, preferably not more than $\frac{1}{16}$ in. above the surface.

10.12 *Chipping and Gas-Gouging of Welds.* The chipping at the root of welds and chipping of welds to remove defects may be performed with a round nosed tool, or by gas-gouging.

10.13 *Flat Tank Bottoms.* The bottom plates, after being laid out and tacked, shall be joined by welding the joints in a sequence which the contractor has found to result in the least distortion due to shrinkage of the welding, and to provide, as nearly as possible, a plane surface.

10.14 *Tank Shell.*

(*a*) On vertical joints, weld metal, for any of the passes, may be applied with either upward or downward progress of the welding,

provided the manufacturer shall have qualified the direction he selects for each pass in his procedure qualification and operator tests.

(*b*) The shell plates shall be joined by welding the joints in a sequence which the contractor has found to result in the least distortion due to shrinkage of the welding and which will avoid kinks at the vertical joints.

10.15 *Matching Plates.*

(*a*) *Lap Joints:* At all lap joints, the plates shall be held in close contact during the welding operation. The separation shall be not more than $\frac{1}{16}$ in. The size of the fillet shall be increased by the amount of the separation.

(*b*) *Butt Joints* (*primary stress*): In butt joints subject to primary stress from weight or pressure of tank contents, the adjoining plates shall be accurately aligned and retained in position during the welding operation, so that in the finished joint, the center lines of adjoining plate edges shall not have an offset from each other, at any point, in excess of 10 per cent of the plate thickness (Use the thickness of the thinner plate if of different thicknesses.) or $\frac{1}{16}$ in., whichever is larger.

(*c*) *Butt Joints* (*secondary stress*): In butt joints subject to secondary stress, the adjoining plates shall be accurately aligned and retained in position during the welding operation, so that in the finished joint, the thinner plate, (if one is thinner than the other), or either plate, (if both plates are of the same thickness), shall not project beyond its adjoining plate by more than 20 per cent of the plate thickness (Use the thickness of the thinner plate if of different thicknesses.) with a maximum of $\frac{1}{8}$ in.

10.16 *Grouting Column and Riser Bases.* After the tank has been completely erected and trued up, any space which may exist between column and riser bases and foundations shall be thoroughly wetted and filled with a 1:1 cement-sand grout forced under the base plates and filling the space completely. The materials and labor for the grouting are to be furnished by the contractor.

10.17 *Sand Cushion.* A sand cushion not less than 1 in. in thickness shall be provided under riveted flat tank bottoms on concrete slab foundations. The sand shall be furnished at the site by the purchaser and spread by the contractor. In the case of both riveted and welded tanks, after the tank has been completely erected, any space which may exist between the tank bottom and the concrete foundation, and extending for a distance of at least 18 in. inside the tank shell, shall be filled with a 1:1 cement-sand grout forced under the bottom so as to fill completely the space. The top of the founda-

tions shall be thoroughly saturated with water before the grout is poured. The materials and labor for grouting are to be furnished by the contractor.

10.18 *Cleaning Up.* Upon completion of the erection, the contractor shall remove or dispose of all rubbish and other unsightly material caused by his operations and shall leave the premises in as good a condition as he found them.

Section 11—Inspection

11.01 *Riveted or Welded Tanks.* The purchaser may, if he so specifies, require mill and/or shop inspection by a commercial inspection agency, the cost of which shall be paid by the purchaser. Copies of the mill test reports shall be furnished the purchaser if requested.

11.02 *Field Inspection.* It is required under these specifications that the field welding be inspected in the following manner by a qualified welding inspector designated by the purchaser. The contractor shall furnish equipment for trepanning or otherwise obtaining inspection specimens.

The quality of all accessible welds which carry stress from weight or pressure of tank contents shall be inspected by sectioning methods.

11.03 *Application.* The examination of the welded structure by sectioning methods is more satisfactory when applied to butt joints, which makes it highly desirable that the shell of the all-welded tank be constructed with butt joints throughout. While it is possible to cut sections from lap welded joints, it will require twice as many sections on the double-fillet-welded joints to obtain as many sections per unit length of joint as for butt joints.

11.04 *Sectional Specimens.*

(*a*) Sectional specimens are segments cut from the welded joints with a cylindrical cutting tool or spherical saw, which removes a portion of the plate bounding the welded joint and exposes thereon two (2) cross-sections of the weld. The segments must expose the full cross-section of the welded joint.

(*b*) Segments cut with a circular tool are called trepanned plugs; segments cut with a spherical saw are called spherical segments.

11.05 *Number and Location of Test Segments.*

(*a*) Sectional testing shall be confined to tank shell joints, particularly those subject to primary stress from weight or pressure of tank contents. It need not be applied to flat tank bottoms resting directly on grade or foundation, nor to the welds between

flat tank bottoms and the first ring of tank shell, nor to the welds connecting top curb angle to shell and to roof, nor to a roof not in contact with the stored liquid, nor to welds connecting manholes and other accessories.

(*b*) Joints subject to primary stress from weight or pressure of tank contents shall have one segment cut from the first 10 ft. of completed joint welded by each operator. Thereafter, cut one segment approximately every 100 ft. of joint. In cylindrical tank shells, cut at least one segment from the vertical seams of each shell ring. It is permissible to qualify two operators' work with one segment if they weld opposite sides of the same seam. When a segment of this type is rejected, it must be determined whether one or both operators were at fault, by subsequent tests of the individual operator's work.

(*c*) Joints subject to secondary stress shall have one segment cut from approximately every 200 ft. of joint, with at least one segment from each circumferential joint of cylindrical tank shells.

(*d*) It is to be recognized that the same welding operator may, or may not, weld both sides of the same joint. Insofar as possible, an equal number of segments shall be cut from the work of each operator welding on the tank, except that this requirement shall not apply where the length of joint welded by an operator is very much less than average.

(*e*) For lap-welded joints, two segments shall be removed at each test point, one from each fillet weld. They shall not be taken on the same center line, but shall be offset approximately 3 in. parallel to the axis of the joint.

(*f*) Spherical segments shall be cut from the outside of the tank shell for butt-welded joints and from the welded surface for lap-welded joints.

(*g*) The location for cutting the test segments may be determined by the purchaser's inspector.

(*h*) Test segments shall be taken as the work progresses, as soon as practicable after all the joints accessible from one scaffold position have been welded.

11.06 *Size of Sectional Segments.*

(*a*) The width or diameter of the segment shall be not less than the width of the finished weld plus $\frac{1}{8}$ in. with a minimum of $\frac{1}{2}$ in.

(*b*) The segments shall be removed on the center of the weld in such a manner that at least $\frac{1}{16}$ in. of parent metal will be removed with the segment on each of the two sides.

11.07 *Preparation of Sectional Segments.*

(*a*) Sectional segments may be etched for inspection by any of the following methods.

(1) Without requiring any finishing or other preparation, place in boiling 50 per cent solution of muriatic (hydrochloric) acid until there is a clear definition of the structure of the weld. This will require approximately 30 min.

(2) Grind and smooth segments with emery wheel and/or emery paper and then etch by treating with a solution of one part ammonium persulfate and nine parts of water by weight. The solution should be used at room temperature and should be applied by vigorously rubbing the surface to be etched with a piece of cotton kept saturated with the solution. The etching process should be continued until there is a clear definition of the structure of the weld.

(3) Grind and smooth segments with emery wheel and/or emery paper and then etch by treating with a solution of one part of powdered iodine (solid form), two parts of powdered potassium iodide, and ten parts of water, all by weight. The solution should be used at room temperature and should be brushed on the surface to be etched until there is a clear definition of the structure of the weld.

(*b*) To preserve the appearance of the etched segments they should, after etching, be washed in clear water, the excess water removed, then immersed in ethyl alcohol, and dried. The etched surfaces may then be preserved by coating with a thin, clear lacquer.

11.08 *Inspection of Sectional Segments.*

(*a*) The etched segments shall be examined to ascertain the extent of weld defects, such as gas pockets, slag inclusions, lack of fusion, undercutting and cracks.

(*b*) For all welds, the etched surfaces of the segments shall show complete fusion between the weld metal and the base metal within the depth of the weld required for the applicable joint. There shall be no cracks in any weld.

(*c*) Slag inclusion is permissible when it occurs between layers of the weld, is substantially parallel to the plate surface, and is equal to not more than one-half the width of the weld metal; and when it occurs across the thickness of the plate and is equal to not more than 10 per cent of the thickness of the thinner plate.

(*d*) Gas pockets are permissible that do not exceed $\frac{1}{16}$ in. in greatest dimension and when there are no more than six gas pockets of this maximum size per square inch of the weld metal and where the com-

bined areas of a greater number of pockets do not exceed 0.02 sq.in. per square inch (2 per cent) of weld metal.

(*e*) For butt and lap joints subject to primary stress from weight or pressure of tank contents, there shall be complete penetration and no undercutting.

(*f*) For butt joints subject to secondary stress, penetration is only required within the limits established by Section 8.04. A maximum undercut of $\frac{1}{32}$ in. at each edge of the weld may be permitted, provided the unwelded gap plus the undercut shall not reduce the thickness of the joint more than $\frac{1}{3}$ of the thickness of the thinner plate joined.

(*g*) For lap joints subject to secondary stress, an undercut $\frac{1}{32}$ in. deep, measured along either leg, and lack of penetration at the root of $\frac{1}{16}$ in., measured along either leg, or the throat, shall be the maximum permitted, provided the size of the weld is increased to compensate for the deficiency in root penetration.

(*h*) Where a defective segment is located, additional segments shall be cut from the same operator's work 2 ft. on each side of the defective segment, wherever the joint length will permit. If additional defective segments are found, then more segments shall be cut at intervals of 2 ft. on the same operator's work until the limit of the defective welding has been definitely established; or the contractor may proceed to replace all the welding done by that operator without cutting out additional segments.

(*i*) Defects in welds shall be removed by chipping or by gas-gouging from one or both sides of the joint, as required, and re-welding. Only sufficient cutting out of defective joints is to be required as is necessary to correct the defects. Replacement welds in joints shall be checked by repeating the original test procedure.

11.09 *Method of Closing Openings from Which Segments Have Been Removed.* Subject to the stated limitations, openings may be closed by any of the following methods:

(*a*) Openings cut with a spherical saw: For butt-welded joints place a backing-up plate where necessary on the inside of the tank shell over the opening. For lap-welded joints the parent plate opposite the weld will usually serve as a backing-up plate. The opening shall be completely filled with weld metal. Where fillet welds are cut, they shall be rebuilt.

(*b*) Trepanned plug openings for joints subject to secondary stress only: Insert a disc in the hole, which disc shall have a fairly close fit in the hole, and shall have a thickness at least $\frac{1}{8}$ in. less than the thickness of the thinner plate at the joint. The disc shall be

inserted in a mid-position between the surfaces of the thinner plate for butt-welded joints and in a mid-position of the continuous plate for lap-welded joints. Both sides of the disc shall be welded over completely, fusing the circular edges of the disc into the plate material and making the surfaces of the weld substantially flush with the plate surfaces. Rebuild fillet welds where cut.

(*c*) Trepanned plug openings for joints subject to primary or secondary stress and where the thickness of the thinner plate at the joint is not greater than $\frac{1}{3}$ of the diameter of the hole: Place a backing-up plate on the inside of the tank shell over the opening. Fill the hole completely with weld metal applied from the outside of the tank shell. Rebuild fillet welds where cut.

(*d*) Trepanned plug openings for joints subject to primary or secondary stress and where the thickness of the thinner plate at the joint is not less than $\frac{1}{3}$, nor greater than $\frac{2}{3}$, the diameter of the hole: Fill the hole completely with weld metal applied from both sides of the tank shell. Rebuild fillet welds where cut.

(*e*) Trepanned plug openings for butt-welded joints subject to primary or secondary stress and where the thickness of the thinner plate at the joint does not exceed $\frac{7}{8}$ in.: Chip a groove on one side of the plate each way along the seam from the hole. The groove at the opening shall have sufficient width to provide a taper to the bottom of the hole, and the length of the groove on each side of the opening is to have a slope of 1 to 3. Using a backing-up plate on the side opposite which the chipping is done, or a thin disc (not over $\frac{1}{8}$ in. thick) at the bottom of the hole, fill the groove and the hole with weld metal.

(*f*) Trepanned plug openings for butt-welded joints subject to primary or secondary stress and for plates of any thickness: Chip a groove on both sides of the plate each way along the seam from the hole. The groove at the opening shall have sufficient width to provide a taper to the middle of the plate, and the length of the groove on each side of the opening is to have a slope of 1 to 3. Place a thin disc (not over $\frac{1}{8}$ in. thick) in the hole at the middle of the plate and fill the grooves and the hole on both sides with weld metal.

11.10 *Record of Segments.*

(*a*) The segments after removal, shall be properly stamped or tagged for identification; and, after etching, kept in proper containers, with a record of their place of removal as well as of the welding operator who performed the welding. A record shall be made by the inspector, of all segments, with their identification marks on a developed shell-plate diagram.

(*b*) After the completion of the structure, the segments may be retained by the purchaser, if he so desires; otherwise they may be discarded.

11.11 *When Inspected.* It is required that the inspector be on the job before any welding is done, at which time the welding operators shall be qualified or their credentials accepted, after which welding may proceed. It is recommended that the inspector make the examination of the trepanned plugs immediately after the first vertical joints are welded to prevent a large amount of unacceptable welding being done which would later have to be removed. The inspector need, however, stay only until he is satisfied that acceptable work is being done, after which he may leave and then return when the welding to be inspected has been completed so that he may complete his inspection of the welds. The inspector shall, however, visit the job at least once during each month that the work is in progress.

11.12 *Who Pays for the Field Inspection.* The purchaser shall pay the cost of the field inspection. If the inspector is required to work a greater number of days, because of repeated tests due to faulty work or of additional trips to reinspect rejected work, the contractor shall pay the increased cost of such inspection over what it would have been if there had been no faulty or rejected work. The inspector shall report to the purchaser any instances where the contractor fails to repair any rejected work which controversy shall be settled between the purchaser and the contractor in a manner satisfactory to them. Upon completion of the work the inspector shall send a written acceptance of the workmanship to the purchaser and at the same time shall send a copy of such acceptance to the contractor.

Section 12—Testing

12.01 *Riveted Flat Bottoms.* Riveted flat tank bottoms shall be tested on horses before being lowered to the grade. Sufficient water shall be placed in the bottom to cover the bottom angle. Any leaks shall be corrected by calking or redriving rivets, if necessary.

12.02 *Welded Flat Bottoms.* After the bottom has been completely welded, including the attachment of the first course shell sheet, water to be supplied by the purchaser, may be pumped underneath the bottom, maintaining a head of at least 6 in. of liquid, by holding that depth around the edge of the bottom, inside a temporary dam; or the joints may be tested with a suitable material such as strong soap solution or linseed oil under air pressure or vacuum;

or the joints may be tested by the magnetic dust method. The bottom shall be made entirely tight to the satisfaction of the purchaser's inspector.

12.03 *General.* After the tank is completed, but before it is painted, it shall be filled with water furnished at the tank site by the purchaser at proper pressure to fill the tank to the maximum working water level. Any leaks which are disclosed in this test in either the shell, bottom, or roof (if the roof contains water) shall be repaired by either calking for riveted construction or by drilling, chipping or gas-gouging out any defective welds and then rewelding for welded construction. No repair work shall be done on any joint unless the water in the tank is at least 2 ft. below the point being repaired.

12.04 *Disposal of Test Water.* The purchaser shall provide means for disposing of test water up to the tank inlet or drain pipe.

Section 13—Field Painting

13.01 *General.* After completion of a satisfactory test, the tank shall be painted or coated in accordance with the purchaser's specifications. See Sections A-7 to A-11 inclusive for recommended paints and methods of protecting the tank against corrosion.

13.02 *Workmanship.* All painting shall be done in a workmanlike manner. The surfaces shall be cleaned of foreign material and if no shop paint has been applied, the steel shall be wire-brushed before applying the first coat of field paint. If the purchaser specifies that the mill scale be removed, and if pickling or sandblasting is not specified, it shall be understood that the rust and loose mill scale shall be removed by wire brushing only.

13.03 *Painting.* After cleaning the work shall be given the required number of coats of paint or other materials as specified by the purchaser and in the manner recommended by the manufacturer of such materials. Sufficient time shall be allowed for each coat to dry thoroughly before the following coat is applied.

13.04 *Aluminum Paint.* Where aluminum paint is specified for the exterior of the tank, the contractor shall apply the paint either by brush or spray, at his option, unless the manner of application is specified by the purchaser. The paint shall be applied as smoothly as possible.

Appendix

A-1 *General.* This appendix contains recommendations which are believed to represent good practice but they are not to be considered as requirements of the specifications.

A-2 *Information to Be Furnished by Purchaser for an Elevated Tank.*

1. Capacity.
2. Height to bottom capacity level or to overflow.
3. Roof pitch and projection at eaves (unless the purchaser wishes to leave to the contractor the selection of proper and appropriate dimensions).
4. The range of head, if special range is required.
5. Type of joint construction, whether riveted or welded, if there is a preference.
6. Diameter and kind of riser pipe.
7. The desired time for completion.
8. Location of site.
9. Type of road available for access to the site and whether public or private.
10. Name of and distance to nearest town.
11. Name of and distance to nearest railroad siding.
12. Availability of electric power, who furnishes it, at what charge, if any, what voltage and whether direct or alternating current. If a.c., what cycle and phase.
13. Availability of compressed air, pressure, volume and charge, if any.
14. Safe bearing value of soil.
15. Corrosion allowance to be added: (a) to parts in contact with water, (b) to parts not in contact with water.
16. Safety cages required on ladders, if any.
17. Number and location of pipe connections, and kind and size of pipe to be accommodated.
18. Overflow, stub or to ground and size of pipe.
19. Is steel to be sand, grit or shot blasted, pickled or otherwise cleaned of mill scale?
20. Kinds of paint or protective coatings and number of coats: (a) inside surfaces, (b) outside surfaces.
21. Specifications for any additional accessories required.

A-3 *Information to Be Furnished by Purchaser for a Standpipe or Reservoir.*

1. If a standpipe, capacity and maximum height of water.

2. If a reservoir, capacity and diameter.

3. Roof pitch and projection at eaves (unless the purchaser wishes to leave to the contractor the selection of proper and appropriate dimensions).

4. Type of joint construction, whether riveted or welded, if there is a preference.

5. The desired time for completion.

6. Location of site.

7. Type of road available for access to the site and whether public or private.

8. Type, thickness and kind of support of roof, if required.

9. Name of and distance to nearest town.

10. Name of and distance to nearest railroad siding.

11. Availability of electric power, who furnishes it, at what charge, if any, what voltage and whether direct or alternating current. If a.c., what cycle and phase.

12. Availability of compressed air, pressure, volume, and charge, if any.

13. Safe bearing value of soil.

14. Corrosion allowance to be added: (a) to parts in contact with water, (b) to parts not in contact with water.

15. Safety cages required on ladders, if any.

16. Number and location of pipe connections and kind and size of pipe to be accommodated.

17. Overflow, stub or to ground and size of pipe.

18. Is steel to be sand, grit or shot blasted, pickled or otherwise cleaned of mill scale?

19. Kinds of paint or protective coatings and number of coats: (a) inside surfaces, (b) outside surfaces except underside of bottom, (c) underside of bottom.

20. Specifications for any additional accessories required.

A-4 *Information to Be Furnished by Bidder for an Elevated Tank.*

(*a*) A drawing showing the dimensions of the tank and tower including the tank diameter, the height to lower and upper capacity levels, the sizes of principal members and the thickness of plates in all parts of the tank and tower. Also, if required by the purchaser, a tabulation showing the rivet data for each type of riveted joint and the welding data for each type of welded joint.

(*b*) The number, names and sizes of all accessories included.

A-5 *Information to Be Furnished by Bidder for a Standpipe or Reservoir.*

(*a*) A drawing showing the dimensions of the standpipe or reservoir

including the diameter, shell height, and the thickness of plates, the type and thickness of the roof, the thickness of the bottom interior and sketch plates, and the sizes or weights of structural members; also, if required by the purchaser, a tabulation showing the rivet data for each type of riveted joint and the welding data for each type of welded joint.

(*b*) The number, names and sizes of all accessories included.

A-6 *Drawings.* After award of a contract, the contractor shall prepare foundation plans and detail drawings, such drawings to be submitted to the purchaser for approval before proceeding with any fabrication. Unless prohibited by the purchaser, the steel may be ordered from the design drawings, and if any changes in capacity or governing dimensions are made by the purchaser which affect the steel ordered, any loss to the contractor shall be assumed by the purchaser. In the preparation of drawings for welded tanks, standard welding symbols as recommended by the American Welding Society shall be used.

A-7 *Cleaning and Shop Painting.* The purchaser shall specify the method of cleaning the steel and the type or make of paint or coating material to be used. The following alternative methods are believed to represent good practice:

(*a*) Plates and structural shapes may be sand, grit or shot blasted, or pickled, to remove the mill scale, after which they shall receive a shop coat of an inhibitive steel primer on all parts except contact surfaces and edges to be welded.

(*b*) Plates and structural shapes may be dehydrated with the oxy-acetylene torch, removing such mill scale as becomes loosened with one application of the torch followed by wire brushing, after which all parts shall be given one coat of inhibitive primer while the parts are still warm, except contact surfaces and edges to be welded.

(*c*) Plates and structural shapes may be fabricated and shipped to the erection site without paint, and then shall be erected and allowed to weather until the mill scale is loosened from both interior and exterior surfaces, after which the steel shall be dried, wire brushed and painted with one coat of an inhibitive steel primer on all surfaces. *Note:* This method may be objectionable because of the possibility of mill scale being carried into the distribution system.

(*d*) If no special cleaning is desired, the steel may be painted with an inhibitive primer directly over the mill rolled surfaces, after removing such rust, loose scale and other foreign material as may be

removed by wire brushing. The contact surfaces or welding edges should not be painted.

A-8 *Field Painting.* After the tank is completely erected, and after it has been tested, interior and exterior surfaces should be cleaned of dirt, rust or foreign material, the welding edges or rivet heads wire brushed, and then any abraded spots and all accessible surfaces which were not painted in the shop shall then be spot coated with the inhibitive primer used in the shop, after which the interior and exterior surfaces shall be given the number of coats of paints of the makes or types, and colors specified by the purchaser.

A-9 *Kind of Paint.* It is recognized that various types and kinds of paint have in great measure demonstrated their suitability for water works use and that no one type or kind is universally applicable. The following are indicative of what is believed to be best practice at present.

(*a*) *Interior Paint:* All interior surfaces of the tank shall be given one coat of interior first coat paint, either in the shop or in the field.

After erection the first coat, if applied in the shop, shall be retouched and all seams and other surfaces not painted in the shop, shall be painted with the same paint.

After the first inside paint coat has thoroughly dried, all inside surfaces shall be given a coat of inside second coat paint in accordance with the formula in Section A-10.

(*b*) *Outside Surfaces:* All outside surfaces, including all structural members with the exception of the underneath surface of flat bottoms, shall be given one coat of outside first coat paint in the shop or in the field.

After erection, if the first coat was applied in the shop, it shall be retouched with the same paint covering any abraded spots or surfaces which were not painted in the shop, after which all outside surfaces shall receive a second coat of either outside black paint or outside aluminum paint.

(*c*) *Underside of Flat Tank Bottoms:* The underside of flat tank bottoms shall be painted in the field If the bottom is riveted, it shall be painted after it is tested and while it is on horses. If welded, the bottom plates may be painted and, after drying, placed in position on the grade before being welded. The underneath surfaces of bottoms shall be painted with black asphalt paint.

A-10 *Types and Kinds of Paint.* The following paints are suggested as suitable for painting tank structures.

All ingredients of the following paints shall be in accordance with

applicable current specifications of the American Society for Testing Materials.

(*a*) *Paint Number 1—Inside First Coat:*

Paste Red Lead	100 lb.
Boiled Linseed Oil	1.625 gal.
Fine Litharge	10 lb.

Beat up the litharge in one quart of boiled oil and add to paint. Makes about 4.25 gal. of paint.

Paint should be applied without thinning whenever temperatures are high enough to permit thorough, smooth brushing out (about 70° F.). When paint is too thick to brush out smoothly, turpentine may be added, using the minimum amount necessary to secure proper consistency, but in no case to exceed 3 pints of turpentine to 100 lb. of paste red lead. (The above is substantially in compliance with Federal Specifications of the Bureau of Standards TTP-86, adopted September 9, 1939.)

(*b*) *Paint Number 2—Inside Second Coat:*

Same as first with addition of 0.75 lb. of paste lampblack to each batch.

(*c*) *Paint Number 3—Outside First Coat:*

Paste Red Lead	100 lb.
Raw Linseed Oil	2.125 gal.
Drier	1 qt.
Turpentine	1 qt.

Makes about 4.87 gal. of paint.

Note: The above red lead paint formulas are based on paste red lead made by grinding 100 lb. of dry red lead with 8 lb. of linseed oil.

(*d*) *Paint Number 4—Outside Second Coat—Aluminum Finish:*

Pigment—2 lb. Aluminum Paste
Vehicle—1 gal. long oil varnish

The aluminum paste shall comply with the latest revision of A.S.T.M. Specifications D 474 (type A).

Long oil varnish shall comply with the following specifications:

The vehicle shall consist of a long oil varnish, made from ester gum, cumaroneindene, Amberol B1 or F7 or other suitable resins, together with suitable drying oils, and shall fulfill the following requirements:

(1) The varnish shall be clear and transparent.

(2) The viscosity shall be between 0.65 and 1.25 poise at 25°C. (77°F.), corresponding to Tubes B to E of the Gardner-Holdt Air Bubble Viscometer.

(3) The acid number of the vehicle shall be less than 15, based on the non-volatile content of the varnish.

(4) It shall contain not less than 50 per cent by weight of non-volatile oils and gums.

(5) It shall pass a 60 per cent Kauri Reduction Test as described in Federal Specification TT-V-81, paragraph F-2g.

(6) When thoroughly mixed with the aluminum paste specified in the proportion of 2 lb. per gal. of vehicle, the paint shall have good leafing quality, show satisfactory brushing and leveling properties, and shall not break or sag when applied to a vertical, smooth, steel surface.

(7) The paint shall set to touch in not less than 1 hr. nor more than 6 hr. and dry hard and tough in not more than 24 hr. at a temperature of 20°C. (68°F.) to 30°C. (86°F.).

(*e*) *Paint Number 5—Outside Second Coat—Black Color:*

The paint furnished shall comply with the United States Department of Commerce Circular of the Bureau of Standards, No. 94, U. S. Government Master Specification 14b for ready-mixed paint.

(*f*) *Paint Number 6—Asphalt Varnish for Underneath Surface of Flat Bottoms:*

Paint furnished shall comply with U. S. Government Specification for Asphalt Varnish, Federal Specification Board Standard Specification 19, revised January 2, 1923.

A-11 *Electric* (*Cathodic*) *Protection of Tank Interiors.* The electrolytic method of protecting water tank interiors appears to have merit although at this date (April, 1940) it has not been used a long enough time or in enough different localities to justify definite conclusions.

A-12 *Foundations—General.* The foundations for elevated tank structures are of considerable importance, because any unequal settlement changes considerably distribution of stresses in the structure and may cause leakage or buckling of the plates. The following is recommended practice with regard to foundations.

A-13 *By Whom Designed.* The contractor shall furnish foundation plans based on an assumed soil pressure of 4,000 lb. per sq.ft. for usual conditions. If purchaser's soil bearing value is lower than 4,000 lb. per sq.ft., special foundations shall be designed jointly by the purchaser and contractor or their representatives, the purchaser to bear the cost of any required field tests. Foundations shall be installed by the purchaser who shall furnish all materials except anchor bolts. The earth around the foundations shall be regraded sufficiently to permit efficient work during erection.

A-14 *Soil Bearing Value.* Unless the purchaser has knowledge of the soil at the tank site, he shall conduct suitable tests to determine the character and safe bearing value of the soil and its condition as regards homogeneity, freedom from old excavation or fill or faults of any kinds

The following are average soil bearing values.

1. Quick sand and wet alluvial soils, ½ ton. per sq.ft.
2. Soft wet clay or sand, 1 ton per sq.ft.
3. Ordinary clay, dry sand mixed with clay, moderately dry sand or firm dry loam, 2 tons per sq.ft.
4. Compact coarse sand or hard clay or gravel, 3 tons per sq.ft.
5. Hard pan or shale in horizontal layers and dry, 5 tons per sq.ft.
6. Hard rock, 20 tons per sq.ft.

A-15 *Riser Foundations.* Foundations for the riser pipe shall be provided with a tunnel of adequate size to accommodate the base ells for the piping specified by the purchaser. The opening shall be covered with a sufficient thickness of concrete either properly reinforced to support the load on the riser pipe bottom, or steel beams may be provided for this purpose. The foundation beneath the tunnel shall be adequately reinforced. The contractor shall furnish plans for the riser foundation.

A-16 *Column Foundations.* Column foundations may be of any suitable shape and may be either plain or reinforced.

A-17 *Concrete Design and Materials.* The design of the concrete foundations, the specifications for the cement and aggregate and the mixing and placing of the aggregate shall be in accordance with the joint code of Building Regulations for Reinforced Concrete, Report of Committee E1 of the American Concrete Institute and Concrete Reinforcing Steel Institute Committee on Engineering Practice.

A-18 *Detail Design of Foundations.*

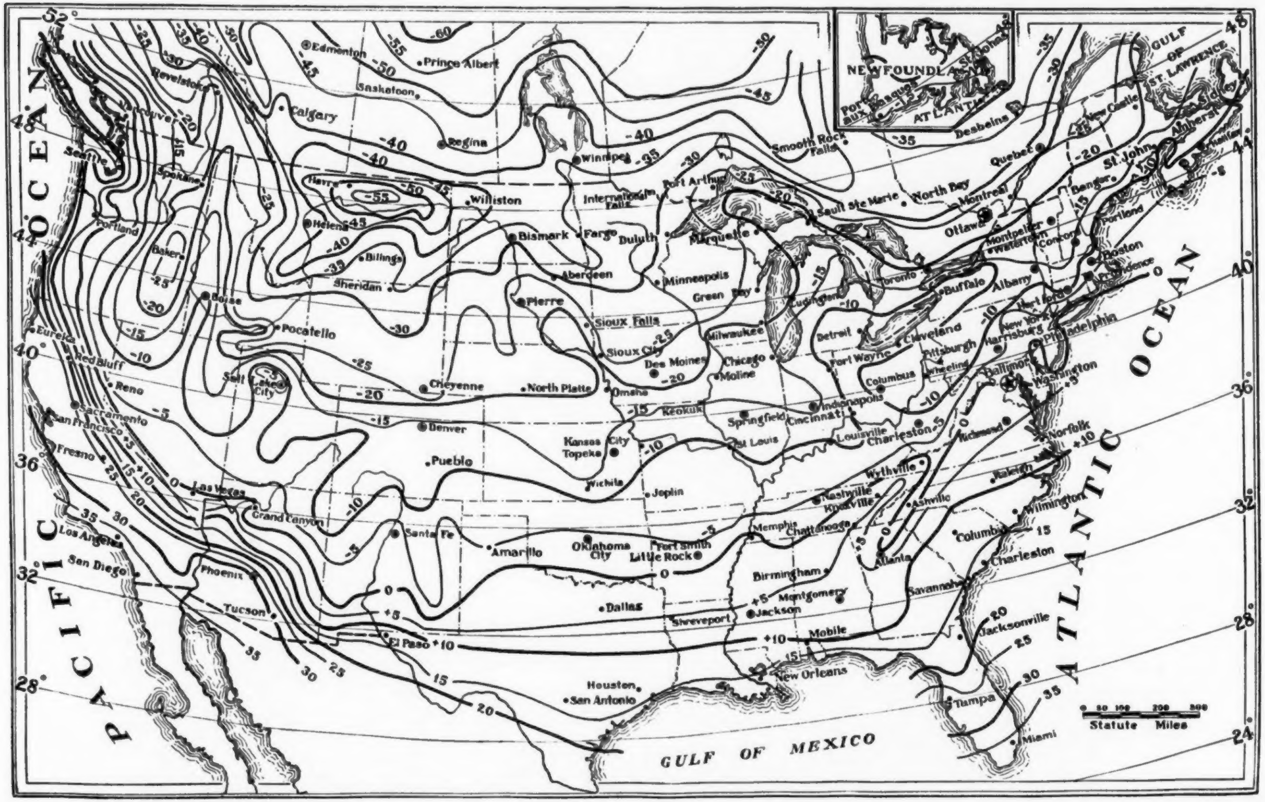
(*a*) *Batter:* For battered columns without bottom struts the axis of column foundations shall have the same batter as the column. For battered columns with bottom struts attached to columns and for vertical columns the axis of the foundations shall be vertical.

(*b*) *Height Above Ground:* The tops of the concrete foundations shall be at least 6 in. above the ground.

(*c*) *Minimum Depth:* The minimum depth of foundations shall be determined from the minimum average one-day mean temperature chart, as follows:

For minimum temperatures over 32°F.—3 ft. 0 in.

For minimum temperatures under 32° to and including +10°F. —3 ft. 6 in.



Isothermal Lines—Lowest One-Day Mean Temperatures (Compiled from United States Weather Bureau Records up to 1925); reproduced by permission of the Associated Factory Mutual Fire Insurance Companies

For minimum temperatures under +10° to and including −10°F.—4 ft.

For minimum temperatures under −10° to and including −20°F.—4 ft. 6 in.

For minimum temperatures under −20° to and including −30°F.—5 ft. 0 in.

For minimum temperatures under −30° to and including −40°F.—5 ft. 6 in.

For minimum temperatures under −40°—6 ft. 0 in.

The above minimum depths refer to the depth of the base below the ground line. They should be increased in localities where soil or other factors are favorable for deep frost penetration.

A-19 *Size of Top.* The tops of foundations shall project at least 3 in. beyond the column or riser base plates. The top corners shall either be neatly rounded or finished with suitable bevel.

A-20 *Pouring.* The riser and column piers shall each be poured monolithically, without any interruption of sufficient duration to permit the concrete partially to set. If it is necessary to pour piers in more than one pour, a sufficient number of dowels shall be used to transmit all specified loads and horizontal wind load shears.

A-21 *Finish.* The top portions of piers to a level 6 in. below the proposed ground level shall be finished smooth. Any small holes may be troweled over with mortar as soon as possible after the forms are removed.

A-22 *Design of Foundations Without Reinforcement.* Unreinforced piers shall have a vertical portion at the base at least 6 in. in height and shall be designed according to the following allowable unit stresses based on the 28-day compressive strength of the concrete used.

Maximum tension in bending, 1.5 per cent of 28-day compressive strength.

Maximum shear, 2 per cent of 28-day compressive strength.

Maximum bearing, 20 per cent of 28-day compressive strength for dead and live loads or 25 per cent for dead and live loads combined with wind load.

final legislative act, not confiscating property in a permanent sense, as water company could recoup losses through a temporary increase if final prescribed rates proved to be higher than temporary rates. No limitations on duration or amount of temporary rates if recoupment was afforded. Regulatory commissions should be fair in their administration of temporary rates if they are to offer a solution for the problem of rate regulation. They should give careful consideration to established law in applying temporary rates, and temper their political views with a facing of facts in the record. In few states having temporary rate making not enough thought has been given to mechanics by which a utility may recoup any taking of its property by a non-compensatory temporary rate. Legislature can abolish a law enacted leaving utility powerless to collect losses. In some states, e.g. Pa. temporary rates not on a uniform basis, may be confiscatory even with recoupment. Pa. statute allows 5% on physical property only. Exclusion of indirect costs in reaching a rate base new in valuation proceedings. Idea of temporary rates appears good, probably a better solution than prudent investment theory. Where temporary rate is compensatory, utility will save a great deal of money in useless litigation, engineering, and accounting studies. Temporary rates at least offer a new approach and partial solution of difficult problem. If properly administered, they would speed up process of rate making and eliminate much useless effort and expenditure on part of both the regulatory commissions and the utilities.—*Samuel A. Evans.*

Beaver Valley Water Co. v. Denis J. Dricoll et al. Decision U. S. District Court. W. D. Pennsylvania. Pub. Util. Fort. (Dec. 7, '39), P.U.R. **30:** 305. Injunction suit to restrain enforcement of State Com. order temporarily reducing water rates; temporary injunction dissolved and bill of complaint dismissed. Com. initiated inquiry and investigation resulting, in order requiring temporary rate reduction amounting to $29,500 in annual gross revenue. No oral argument or briefs filed with Com. Water company claimed ruling unconstitutional, violating Fourteenth Amendment. Question, whether temporary rate constitutional, if although based solely on depreciated original cost, there is statutory provision for recoupment by utility company if temporary rate is finally determined not to provide fair return, this is issue before the court. Com. fixed temporary rates upon basis of original cost less accrued depreciation, as statute required, allowing 1% more than statutory minimum. Due process clause fully available to company in proceedings to determine final rates. Therefore temporary rate not unconstitutional, since there exists statutory provisions for recoupment of losses suffered by water company, due to temporary losses.—*Samuel A. Evans.*

Borough of Ambridge v. Pennsylvania Public Utility Commission, Before Pennsylvania Superior Court. Court Ruling. Pub. Util. Fort. (Jan. 4, '40), P. U. R. **31:** 50. Appeal by borough from Commission order fixing municipal plant rates for water service outside of borough; reversed, and complaint by customer dismissed. Vital question whether borough supplying water to its own inhabitants from own plant, also furnishing water outside borough (1) may establish a different rate schedule for consumers outside borough, or (2)

may be required by Public Utility Com. to treat outside customers as if whole plant were a private water company, and no segregation of customers for rate making purposes. Borough takes first position, com. and outside consumers second; court concurs in borough's contention. Public Service Company Act of 1913 gave com. no power of supervision or regulation over rates charged by a municipal corporation which furnished water or other public service to its inhabitants or to customers residing outside its limits. But the Public Utility Law of May 28, 1937 stated "that any public utility service being furnished or rendered by a municipal corporation beyond its corporate limits shall be subject to regulation and control by the com. as to rates, with the same force, and in like manner, as if such service were rendered by a public utility." The com. used this section of the act to base its contention as to schedule of rates outside a borough, but treated the whole as one operating private utility. This overlooks the provision giving the com. *any* authority, restricts it to any public utility service *beyond its corporate limits*. Com.'s contention if sustained would give it authority to determine rates within the borough, but it could not enforce them. Court rules statute does not give com. either authority, but only to regulate and control rates beyond borough limits, fair and reasonable rates based on fair value of property used, with just allocation, of plant investment within borough and of operating, maintainance, and depreciation charges. Com. ignored use of facilities and allocated on an arbitrary basis. A third class city fixed rate schedules for consumers inside the city and a higher rate for those outside city. Present Chief Justice laid down following principles: (1) rates of a municipally owned water company are controlled by the courts and not by Public Utility or Service Commissions; (2) applicable to rates outside the city controlled by Public Utility Com., law requiring fair value standard; (3) courts or legislature cannot prohibit a municipal water plant from making a fair return by way of profit on furnishing water; (4) determinarion of fair value of a municipal water system must be made in the same manner as private utility corporations; (5) fair value of plant for rate fixing within city must have plant fair value for outside city deducted; (6) cost of operating plant with all expenses must be proportioned for outside service; (7) city may charge more for water outside its limits—citizens cannot be asked to pay expenses incurred in supplying water outside the city.—*Samuel A. Evans.*

Borough of Auburn v. Auburn Water Co. Pennsylvania Public Utility Commission. Ruling. Pub. Util. Fort. (Jan. 4, '40), P. U. R. **31**: 56. Borough of Auburn complainant, contended franchise provided water company install fire hydrants, city pay $14 annually for first 20, $5 for each additional until such time as revenues equal interest upon cost of plant, then water furnished free to all fire hydrants. Borough maintained after furnishing free water service from '06 to '38, respondent filed a tariff charging $25 annually for each fire hydrant, which is excessive and unwarranted. Respondent maintained free water service cannot be rendered to consumers, even a municipality, and burden of proof in an attack on rates rests with complainant. Three questions involved: (1) whether discriminatory for a public utility to furnish public free fire hydrant service; (2) whether a hearing or not on complaint; (3)

whether or not in a complaint proceeding against the unjustness and unreasonableness of an existing rate of a public utility, burden of proof upon complainant. Public utility law provides utility must adhere to tariffs; in all cases, there must not be any discrimination in any way whatsoever. Hearing granted, complainant may present case. Burden of proof, as to excessive and unjust fire hydrant service rates, upon Borough of Auburn.—*Samuel A. Evans.*

New Jersey Suburban Water Company v. Board of Public Utility Commissioners et al. (New Jersey Court of Errors and Appeals Ruling.) Pub. Util. Fort. (Feb. 15, '40) P. U. R. **31**: 219. Water Company had a 15-year contract, with 10-year option, to sell water to town of Harrison. During option period exercised by Water Company, it petitioned Board of Utility Commissioners for rate increase, board fixed a rate, which was confirmed by N. J. Supreme Court. Two questions: Did Supreme Court erroneously exercise supervisory power? Is fixed rate reasonable? Supreme Court decision reviewed on merits, unwise for Error and Appeals Court to determine status of rate, where it is questionable whether the Supreme Court made same factual finding as Commission. Reproduction cost is a factor in determining fair value for rate making purposes. Value of water company's plant depends upon use and profitableness of present and prospective services rendered. If a main used is twice necessary size, and water company is obligated to use same, it is entitled only to a fair return upon fair valuation of that main. Depreciation starts on a water plant and additions from moment of use. Testimony of competent valuation engineers preferable to mere calculation based on averages and probabilities for rate making purposes. Taxes on property other than that useful to public should not be allowed as operating expenses. Under all circumstances a water company is only entitled to compensation for service rendered. Rate return of $6\frac{3}{4}\%$ was sustained as fair and just in reviewing a commission order. Commission justified in considering comparative rates in similar localities when fixing rates.—*Samuel A. Evans.*

Erratum

On page 943 of the June, 1940, issue of the JOURNAL the word "*thousand*" should be substituted for "*million*" in the item now reading "Ave. revenue *per million cu. ft.*"

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New High Pressure Steel Water Supply Line For Colorado Springs

By E. L. Mosley

COLORADO Springs is one of a small group of American cities that owns and operates its electric, gas and water utilities. The electric and gas properties serving the Pike's Peak Region were purchased in 1925 and subsequently consolidated with the water system, already publicly owned, into one department of public utilities. During the last fifteen years, an extensive rehabilitation and construction program has been carried forward consistently on all of these divisions. The total cost of the program has been in excess of $5,235,000, with $3,000,000, or nearly 60 per cent of that amount, having been allotted to the Division of Water and Water Works.

The water supply has its source on the south, east and north slopes of Pike's Peak. The watershed, 66 square miles in extent, lies at elevations between 6,700 and 14,000 ft. above sea level. The system is a gravity one, and wide seasonal variations in precipitation are compensated for by the maintenance of nine storage reservoirs located at various points on these slopes. They have a total capacity of 12,944 acre-ft. and are situated at elevations ranging from 9,200 to 12,000 ft.

A paper presented on April 23, 1940, at the Kansas City Convention by E. L. Mosley, City Manager, Colorado Springs, Colo.

In addition to storage water, made available when needed from one or more reservoirs, water is taken directly into the transmission system from nine small mountain streams, with pipe line intakes at somewhat lower elevations, but lying within a rigidly patrolled watershed reservation.

The Colorado Springs system serves approximately 40,000 people through 15,300 taps. The average use in 1939 was 230 gallons per capita per day. About $6\frac{3}{4}$ per cent of the services are metered, the remaining consumers being supplied on flat rate schedules, adopted many years ago and still in effect. With the exception of a light chlorination dosage given to meet United States Public Health Service standards, no filtration or other preparatory treatment is now considered necessary.

Since 1892, steel pipe lines, varying in diameter from 14 to 24 inches, have been built as required to meet current demands, so that on January 1, 1940, the capital invested in transmission lines only, including intakes and appurtenant structures, had reached a total of $1,448,474. This sum represents an average cost of $75,441 a mile for the 19.2 mi. of such lines in service on that date. The cost per foot varies from $3.41 for the original 14-inch line built in 1892, to $35.07 for the reconstructed 24-inch pressure line built in 1939.

The first steel pipe line, 2.87 mi. long, was laid in the bottom of a cañon parallel to the lower portion of the "Cog Road," which extends from Manitou Springs to the summit of Pike's Peak. It has a total fall of 2,559 ft. between its upper and lower ends, and is used exclusively as an open end supply line. Built of Matheson joint pipe, it has been utilized for emergency purposes since 1905, when the substitute pressure pipe line, put in operation in February of that year, is occasionally taken out of service for repairs. Although this original steel pipe line is now 48 years old, it is in excellent condition and there is reason to believe that it will satisfactorily serve its purpose, as an auxiliary conduit, for many years to come.

The first high pressure steel pipe line built on the system was constructed in 1903 and 1904 by the Pike's Peak Hydro-Electric Company, a private corporation acting under a 25-year franchise granted by the city in 1898. When placed in operation in 1905, it diverted the water formerly carried by the line installed in 1892; and for the past thirty-five years, it has been used for power generating purposes, as well as for the transmission of the domestic water

supply. At the time it was built, it represented the highest head—2,417 ft.—ever attempted in this country. It was 3.28 mi. long and had an inside diameter of approximately $20\frac{1}{2}$ in. The heaviest pieces of pipe were formed of $\frac{3}{4}$-inch sheet steel rolled into tubular form, with the longitudinal seam riveted to double butt straps, one inside and one outside the pipe. The pipe was made up of 32-foot lengths between flanges, each length being composed of 5-foot sections riveted together.

Before being put in service, the lower section was tested at a pressure of 2,000 lb. per sq.in., the general rule for the entire job being to test at twice the working pressure. The pipe was calculated to have a factor of safety of 3.0 throughout. A good description of this pioneer project is to be found in the May 26, 1906, issue of the *Electrical World*, under the title "Plant of the Pike's Peak Hydro-Electric Company."

After nearly thirty-five years of continuous use, it was taken out of service and abandoned on July 17, 1939, because its carrying capacity had become insufficient to meet peak load demands for both domestic water and power requirements, and because of its structural weaknesses which resulted directly from age. The work of replacing the penstock was finished in 1939. As now constituted, it is 2.96 mi. long, and has an average inside diameter of $22\frac{5}{8}$ in. The available maximum head of 2,421 ft. produces a static pressure of 1,051 lb. per sq.in. at the power house.

For descriptive purposes, the entire project, shown in profile in Fig. 1, can be divided into three general divisions as follows:

(1) The Upper Section, 3,359 ft. long, rebuilt in 1928–1929 by departmental construction crews.

(2) The Middle Section, 7,322 ft. long, contracted for in 1939.

(3) The Lower Section, 4,943 ft. long, also contracted for in 1939.

The Middle and Lower Sections were built as a P.W.A. project, the former being constructed on a new location, about 1,687 ft. shorter in length than the corresponding portion of the old line.

Basis of Design

Selection of the diameter most economical to use was not a difficult task since much pertinent data had been collected over a long period of time. This information covered both water and power demands under all types of load conditions. It was known that the average annual available supply of water flowed at the rate of 11

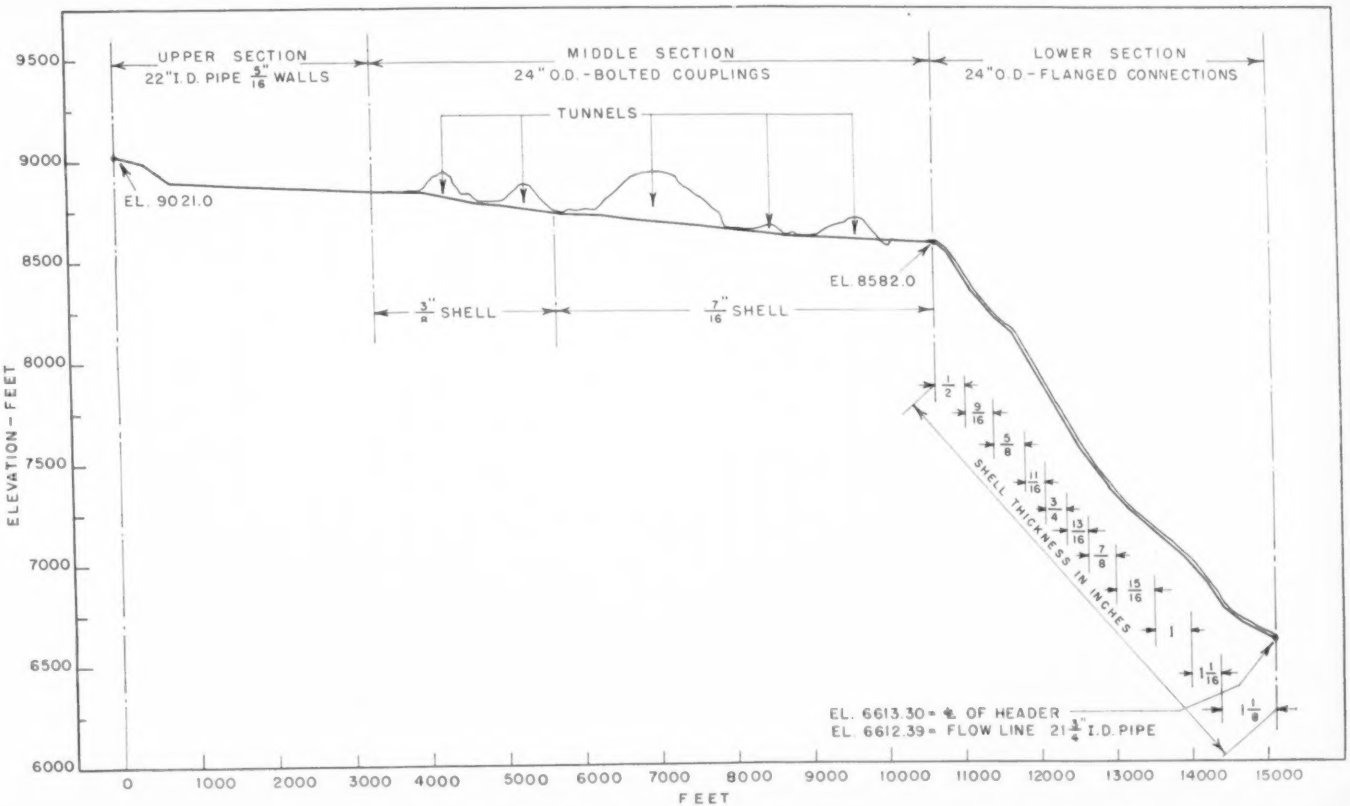

FIG. 1. Profile of Colorado Springs Pipe Line

cu.ft. per sec., and further that, if the new line was to function to the best advantage as a power plant penstock, it would be required to carry, for periods of about 3 hr. per day, at least three times this volume of water without undue friction head losses. The diameter chosen, 24 in. outside diameter and an average of $22\frac{5}{8}$ in. inside diameter, gives velocities of about 4 and 12 ft. per sec. respectively for the volumes indicated. These rates are consistent with good hydraulic practice at the present time.

Table 1, prepared from results obtained by actual tests on March 1, 1940, gives the friction head losses per thousand feet of pipe line for the several volumes given.

TABLE 1

Friction Head Losses, March 1, 1940

FLOW		EFFECTIVE HEAD	FRICTION HEAD LOSS	
			Total	Per 1000 ft.
cu. ft. per sec.	*g. p. d.*	*ft.*	*ft.*	*ft.*
0	0	2,421	0	0
5	3,231,585	2,413	8	0.51
10	6,463,170	2,393	28	1.79
15	9,694,755	2,362	59	3.78
19.60	12,667,813	2,322	99	6.34
25	16,157,925	2,261	160	10.24
30	19,389,510	2,194	227	14.53
35.64	23,034,738	2,110	311	19.91

It is proper to note that the maximum capacity of the line which was replaced by this new project was 19.60 cu.ft. per sec., and that, with this volume of flow, the total friction head loss before replacement was 429 ft., or $4\frac{1}{3}$ times the friction head loss resulting from the same volume of flow under the changed conditions. This increased efficiency was made possible by three factors: (1) a decrease of 0.32 mi., 9.76 per cent, in length; (2) an increase in the smoothness of the pipe interior by the elimination of rivet heads, etc.; and (3) an increase of approximately 2 in. in the average diameter which it was possible to obtain, under existing circumstances, without excessive cost. The Williams-Hazen coefficient of smoothness, C, is approximately 135, even though no special treatment was given to the inner surface of the steel pipe other than two coats of bituminous paint, applied cold before shipment.

Pipe wall thicknesses were computed by the use of "Barlow's" formula, $t = \frac{D \times P}{2 \times s}$, using the following factors in each calculation:

t = Pipe wall thickness in inches.
D = 24 inches, or the outside diameter of the pipe used.
P = The internal pressure in pounds per square inch resulting from the weight of the water column.
s = The unit working stress of 15,000 pounds per square inch or one-half the specified yield point of the steel used.

In order to make the proper provision for surges of short duration, caused by power production units operated in connection with this job, an arbitrary allowance of 264 lb. per sq.in. was added to each controlling pressure calculated directly from the profile. This figure represents 25 per cent of the *maximum* static pressure applied for the full length of the line.

After the various wall thicknesses had been determined as above outlined, the element of irregularity in steel plate rolling practice was eliminated by adding $\frac{1}{16}$ in. to the calculated thickness for each length of pipe. This additional metal also serves as a protection against corrosion, although this particular hazard is a minor one due to the non-corrosive characteristics of the disintegrated granitic material in which the pipe is laid.

Upper Section

The upper section is made up of the following parts:

(1) A reinforced-concrete circular intake tank having a diameter of 150 ft. and an over-all depth of 10 ft. The effective depth between the top of the outlet cone and the spillway is 8.21 ft. resulting in a net capacity of slightly more than 1,000,000 gallons. It was built in 1907 and the present outlet structure was installed in 1928. This tank serves as the focal point for the collection of direct flow and storage water obtained from some 21 sq.mi. of mountain watershed.

(2) The pipe used, outside diameter, $22\frac{5}{8}$ in., was manufactured by the hammer-weld process, with a shell thickness of $\frac{5}{16}$ in. It was furnished in single random lengths with plain ends. The joints are made up of standard and special bolted type sleeve couplings. The special couplings were used at changes in alignment where the deflection angle was in excess of $2\frac{1}{2}$ degrees, the maximum angle allowable when the standard "Style 38 Dresser" coupling is used.

The pipe is laid in a trench cut out of a side-hill bench by hand labor, lack of roads and highways making it impossible to employ even the most common construction equipment usually found on work of this character.

A shut-off valve is installed at each end of the section. Immediately below the valve at the upper end, there is a 4-inch "Crispin" air vent inserted in the line to expedite the filling operation, as well

FIG. 2

FIG. 3

FIG. 2. Cabin Creek Trestle
FIG. 3. Junction, Upper and Middle Sections

as to remove any possibility of collapse which might result from emptying it in too short a time. A steel trestle, 220 ft. long and 60 ft. high, was required to carry this section over the Cabin Creek cañon (see Fig. 2). It was built in 1925, replacing a wooden structure of approximately the same dimensions. The trestle was designed to carry two 22-inch pipe lines, and on it the pipe is supported by semi-circular bent angle saddles placed about 6 ft. apart and

anchored by U-shaped rods bolted through the floor beams. Although the pipe carried by the structure is fully exposed to fluctuating air temperatures at various seasons of the year, the problem of expansion and contraction is not a particularly serious one, since the temperature of the water in the pipe line never falls below 33°F. even when air temperatures reach as low as −30°.

(3) Near the lower end of the section, a 16-inch outside-diameter feeder line, 1,553 ft. long, replaced in 1934, brings the direct flow of Cabin Creek into the main penstock. This line was built of plain-end seamless pipe with bolted-sleeve couplings and having a wall thickness of $\frac{5}{16}$ in. The main-line and feeder-line inlets are set vertically with the top of the inlet cone, placed at some distance above the bottom of the intake basins to prevent fine sand and debris of various kinds from entering the penstock during periods of abnormally large runoff. Prior to changes in the design of these inlets, the needles and nozzles in the power house were frequently damaged by small amounts of sand carried through them at the excessively high velocities which are developed there. At the time this section was built and put in service, the connection between it and the original downhill portion of the penstock was made by means of a special cast steel wye, so designed as to permit construction operations to be undertaken on the middle and lower sections of the replacement work without interrupting service. Although ten years elapsed before further work of replacement became possible, this arrangement proved to be a very valuable one from both the construction and operating points of view.

The pipe on this section was dipped in asphaltic compound before shipment from the mill.

Middle Section

As has already been stated, the middle portion of the project, when rebuilt in 1939, was shortened approximately 1,687 ft., which change necessitated the construction of five tunnels to secure the desired improvement in alignment and grade. The combined length of these tunnels is 3,932 ft., representing nearly 54 per cent of the total length of the section. They are 5 ft. wide, 7 ft. high, unlined and have a minimum depth of 2 ft. of backfill to cover the pipe laid through them. The main purpose of this backfill is to prevent damage from falling rocks which may be dislodged from time to time by weathering conditions. Concrete portals were placed at the ends

of all tunnels to prevent possible cave-ins and slides from damaging the pipe at these points. The pipe in the section has an outside diameter of 24 in. with wall thicknesses of $\frac{3}{8}$ and $\frac{7}{16}$ in. respectively, depending upon pressure conditions. Together with the pipe used for the lower section, it was fabricated from cold-rolled steel plates and electrically welded along the single longitudinal joint by the "Unionmelt" process. The pipe was manufactured from plates having a yield point of not less than 30,000 lb. per sq.in. and an ultimate strength of at least twice that amount.

Current specifications of the American Society of Testing Materials were used except that the radiographic examination of welds was omitted. Each individual piece of pipe was subjected to stress-relieving operations and, before shipment, was hydrostatically tested to approximately 180 per cent of the load it was designed to carry.

The joints are made up with "Style 38 Dresser" couplings having middle rings $\frac{1}{2}$ in. by 8 in., fabricated of heat-treated steel. Although comprising 47 per cent of the total length of the reconstructed project, less than 11 per cent of the total head is developed in this section. Deflections in the joints, both horizontal and vertical, were held to a maximum of $2\frac{1}{2}$ degrees by using short lengths of pipe where necessary to accomplish the result.

A 22-inch flanged cast-steel "Series 300" geared gate valve, weighing approximately three tons, with a cast-steel adapter required to make the change from the 22-inch inside diameter line to the $23\frac{1}{4}$-inch inside diameter line, was used at the junction of this section with the lower end of the upper section (see Fig. 3). One of the minor problems encountered on the job was the transportation of this valve through the tunnels to get it to its proper position in the line. Eleven concrete anchors of about 4 cu.yd. each were placed at critical points to prevent lateral deflection under load.

One of the major construction problems of the job on this section, as well as on the lower section, was that relating to the transportation of supplies, equipment and material to the site of the work. With the exception of the extreme lower end, no portion of either of these sections was accessible to wagons or trucks. The electrically operated cable railway, constructed at the time the original riveted line was built, however, is still in operation as a scenic attraction and this 30-inch gage railway was leased by the contractor for the first six months of the construction period. All of the materials and supplies necessary for the middle section of the line were hauled to

the summit of Mt. Manitou by this means. From that point on, trails and roads were built as required and the ordinary means of transportation by wagon, tractor and narrow gage railway—in tunnels only—was provided. Construction equipment, consisting of tractors, compressors, auxiliary booms, bulldozers, etc. was dismantled at the foot of the Incline Railway and hauled to the top of the mountain on specially built cars in units of 6,000 lb. or less, and there it was reassembled and used until the job was finished.

A steel trestle, $101\frac{1}{2}$ ft. long, similar in design to the Cabin Creek structure, was used to carry the pipe line over Rock Creek Cañon, with the flow line of the pipe approximately 35 ft. above the bed

Fig. 4

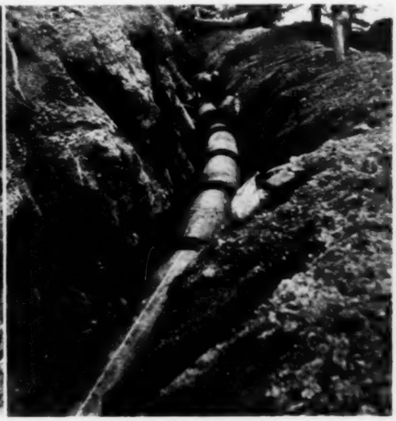

Fig. 5

Fig. 4. Near Lower End, Middle Section
Fig. 5. Rock Cut

of the stream. This structure will also accommodate a second line of the same size when needed in the future.

Lower Section

The lower section portion of the work was the most difficult to construct for a number of reasons, the major one being the extreme steepness of the hillside which necessitated assembling the line virtually on end.

The average over-all grade for the entire length of 4,943 ft. is in excess of 44 per cent, with one continuous run of pipe 870 ft. long built on a grade of 66.48 per cent, or a vertical rise of approximately

8 in. in each 12 in. of horizontal length. The excavation averaged $8\frac{1}{2}$ ft. in depth, much of it in solid rock (see Fig. 5).

For about three-quarters of its length, the pipe trench is located parallel to and only 6 ft. distant from the riveted line which has now been replaced, but which was kept in continuous service during the construction period. One of the provisions of the contract for installation was the requirement that the contractor should carry insurance in the total amount of $100,000 against the possibility of service interruptions, with consequent heavy property damage which might easily result from blasting and other operations vital to the

FIG. 6. Flanged Pipe

successful conduct of the job. Much credit is due the contractor for the manner in which this particular hazard was successfully overcome. The narrow gage railway already mentioned as having been used in connection with the transportation of supplies, equipment and materials on the middle section, was also used to excellent advantage here. The center line was located only 16 ft. from the center line of the new work, making it convenient to transfer individual pieces of pipe and fittings directly from the car to the trench. The advantage of this feature will be apparent when it is remembered that a 17-foot pipe, 24 in. in outside diameter, with a wall thickness of one inch, including two hubbed flanges, weighs 5,900 lb.

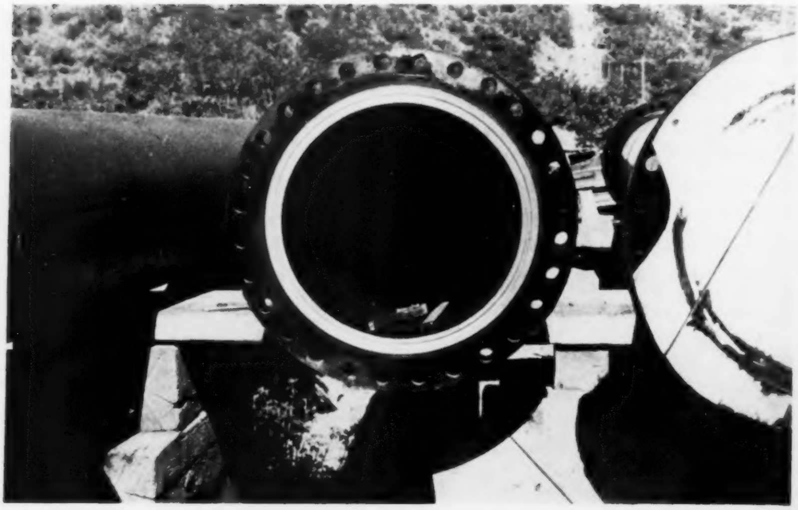

Fig. 7. Van Stone Pipe End

Fig. 8

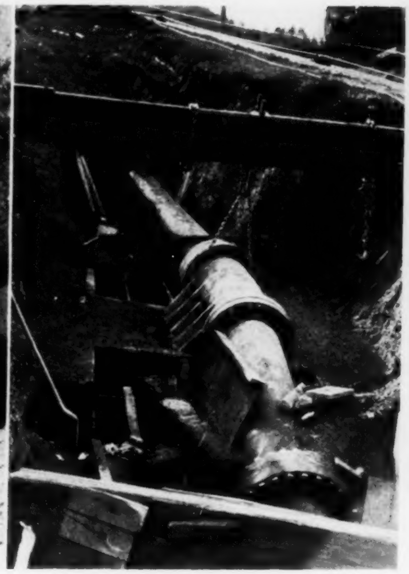

Fig. 9

Fig. 8. Typical Van Stone Joint
Fig. 9. Horizontal Angle, Lower Section

Owing to the high working pressure developed, together with the need for pipe line rigidity and the necessity of securing proper anchorage support at key points, flanged joints were selected as the best type of design available.

The joints are of the "Van Stone" type, with square corner upset pipe ends (see Figs. 7 and 8). They are machined all over to specified dimensions for the various shell thicknesses of pipe, and have gasket grooves in the faces of the upset ends. These gasket grooves are cut to a depth of $\frac{5}{32}$ in. on a gasket circle having a uniform diameter of 24 in. for all pipe and fittings, regardless of other

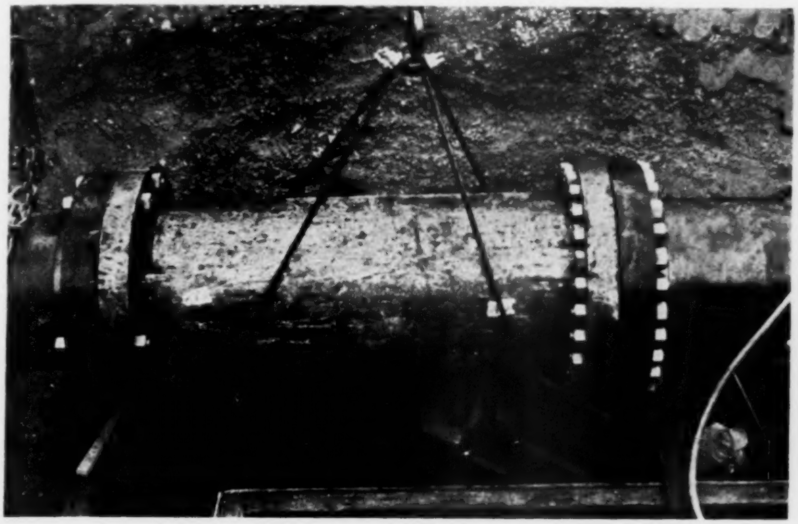

FIG. 10. Anchorage Piece

dimensions. The gaskets used were made of new live rubber, plant vulcanized in endless rings, with a tubular diameter of $\frac{1}{2}$ in., and sized to fit the gasket grooves on the ends of the pipe and fittings described above.

The flanges are of the loose, hubbed type, forged from solid blocks of steel, and the joints were made up by using alloy steel, Grade "C" stud bolts, equipped with semi-finished hexagonal nuts, and having 8 threads to the inch.

The profile of this section of the line has 14 vertical angles in it, 5 of them being located at points where the grade below the angle point is greater than that immediately above it. The change in

grade at all of these points, together with two horizontal angles near the lower end, was accomplished by the use of special flanged cast steel elbows, with center line radii ranging from a minimum of 20 ft. to a maximum of 150 ft. (see Fig. 9). The elbows varied from 4 ft. $2\frac{5}{8}$ in. to 6 ft. $5\frac{1}{16}$ in. in length.

Special flanged cast steel pipe sections, 6 ft. long, with integral bases designed to transmit the load to heavy concrete anchor blocks (see Figs. 10 and 11), were inserted in the line at 7 locations, 5 of them being placed adjacent to and uphill from the 5 special elbows

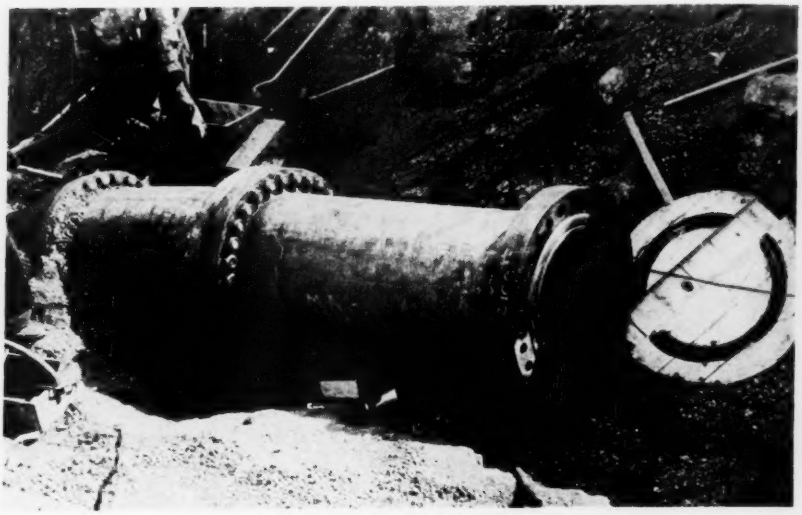

FIG. 11. Anchorage Piece in Place on Lower Section

already mentioned as being required at the points where increases in grade were made.

Seven intermediate anchors were also required at various points (see Fig. 12). These anchors were simply blocks of concrete poured around and over flanged joints in such manner as to transmit the load directly from the flange area to the anchor without special fittings of any kind. These anchor blocks required 365 cu.yd. of concrete. High early strength cement was used in all concrete, so that pipe laying operations would not be unduly retarded while waiting for concrete anchors to set properly.

The pipe was laid uphill, each individual piece being set accurately to line and grade and backfilled before the next joint was made.

Special care was required in the assembly of each joint, the gasket being first set in the gasket groove on the lower side of the joint in rubber cement, with the adjoining pipe being lowered against it and brought to its proper position by means of a tapered metal guide operated by the inspector on the inside of the pipe. After the gasket grooves had been carefully matched, the bolts were drawn up evenly around the entire joint, a pneumatic wrench being used for the purpose. This tool was a most valuable one and enabled the contractor to do a first class job at minimum expense for bell-hole

FIG. 12. Intermediate Anchor

excavation, a very material item on this particular job. The inspector on the inside was equipped with gages and directed operations so that, upon completion of the bolting process, a uniform distance of $\frac{1}{16}$ in. between faces of upset pipe ends was obtained around the entire inside circumference. The excess in cross sectional area of the rubber gasket, after full compression had been obtained in the matched gasket grooves, flowed out between the end faces of the pipe at the joint, separating them slightly.

Since the plans contemplated the renewal of the old power plant header with a new one in the same location, it was necessary to leave

this part of the work until the plant could be shut down, and the replacement made with a minimum loss of plant operation in changing from the old to the new pipe line. This made it necessary for a special closure piece to be inserted between the lower end of the new pipe line and the end of the header at a point just outside the power house. The closure piece is 41 in. long in its fully extended position, is made of heavy forged steel construction, and fabricated in two pieces joined together by a threaded sleeve capable of reducing its over-all length up to a maximum of $3\frac{1}{2}$ in.

FIG. 13. Construction Car

The type of construction permitted the closure to be made without trouble, with the threaded sleeve being tightened after the gaskets had been properly placed, thus bringing the flanged ends into position for bolting in the same manner as all the other joints on the section.

There are two power units, each having a rated capacity of 2,500 kw. at 100 per cent power factor. They are driven by single nozzle, horizontal, undershot, impulse type "Pelton Water Wheels," directly connected to 6,600-volt "Westinghouse" generators, operating at a speed of 900 r.p.m.

Operation and Personnel

Construction work was started December 28, 1938, and the line was tested out and placed in regular service on August 1, 1939.

Since that date, it has been necessary to shut down twice because of leaks resulting from defects not discovered during inspection and testing. Repairs have been made in each case by backing off the flanges, removing the imperfect ends and replacing the joint with a bolted split sleeve, designed especially for the purpose.

With the exception of these interruptions, which occurred on October 25, 1939, and on March 14, 1940, the project has functioned perfectly, with actual operating results better than expected from pre-construction calculations.

The personnel engaged on the project includes: E. L. Mosley, City Manager, Frank O. Ray, City Engineer and C. C. Eastham, Resident Engineer, all of the City of Colorado Springs; B. B. McReynolds, Superintendent, Water Division, and Fred H. Wiley, Superintendent, Electric Division, both of the Colorado Springs Department of Public Utilities; and George M. Bull, Regional Director, V. R. Guthrie, Traveling Engineer and Fred W. Roberts, Resident Engineer-Inspector, as representatives of the Public Works Administration. The Ed H. Honnen Construction Co. of Colorado Springs was the contractor on pipe line and machinery foundations. The Westinghouse Electric and Manufacturing Co. of Pittsburgh installed the power house equipment, and together with the Pelton Water Wheel Co. of San Francisco furnished this equipment. Pipe and fittings were furnished by the Taylor Forge and Pipe Works of Chicago through several sub-contractors: the Omaha Steel Works of Omaha, Neb. furnishing the steel castings; the Bethlehem Steel Co. of Bethlehem, Pa., the alloy steel bolts; and the Dresser Co. of Bradford, Pa., the bolted steel couplings. Valves were purchased from the Rensselaer Valve Co. of Troy, N. Y. and the Pelton Water Wheel Co. Inspection service was rendered by the Robert W. Hunt Co. of Chicago.

Pumping and Related Equipment Used On the Mono Craters Tunnel Project

By S. M. Dunn

THE past year has witnessed the completion, by the Department of Water and Power of the City of Los Angeles, of what is known as the Mono Craters Tunnel. This tunnel (cf. H. L. Jacques, Jour. A. W. W. A. **32:** 43 (1940)) is a major part of works which are being constructed for the purpose of impounding the water flowing in the drainage streams on the west side of Mono Basin and transporting it to the upper end of Owens Valley from which, by means of the Owens River Aqueduct, the city now gets its principal supply. This water, which originates on the east slope of an adjacent part of the Sierra Nevada Mountains, now finds its way into Mono Lake where it disappears by evaporation. The tunnel was made necessary by the existence of high ground intervening between the Mono Basin and the head of Owens Valley. Some of this territory rises to an elevation of more than 9,000 ft. above sea level in the path of the tunnel line. The high ground is surmounted by the Mono Craters, a series of volcanic cones which are said by geologists to be of very recent origin. The truth of the statement is attested to, to some extent, by the fact that there are a number of large hot-water springs in the neighborhood, and also by the fact that in the section of the tunnel, immediately beneath the volcanic cones, the temperature of the water encountered was more than 120 degrees Fahrenheit, and accompanied by a large inflow of carbon dioxide gas.

When the works are completed the supply now being obtained from the Owens River Aqueduct will be augmented by 90 million gallons per day of water of very excellent quality.

A paper presented on April 23, 1940, at the Kansas City Convention by S. M. Dunn, Mechanical Engineer, Bureau of Water Works & Supply, Los Angeles, Calif.

The tunnel is 59,812 ft., or approximately 12 mi., long; is circular in cross section with a diameter of 9 ft. 7 in.; has a slope of ½ ft. per 1,000 ft., and a mean elevation of 7,028 ft. above sea level. The plan adopted for driving the tunnel made use of two working shafts so that six working faces would be available. These shafts have depths of 300 ft. and 890 ft., the shallower being located 12,700 ft. from the east or lower portal, and the deeper, 24,500 ft. from the west or upper portal; these locations having been chosen with accessibility at the surface in view. A third shaft with a depth of 535 ft. was sunk, however, for the accommodation of additional ventilating and unwatering lines when a large inflow of gas and gas-charged water was encountered as the line of the craters was approached. This shaft was located 11,000 ft. from the upper portal.

General Problems

Preliminary test wells drilled along the line of the tunnel had indicated that 65 per cent of the length of the tunnel would lie more than 200 ft. below and 35 per cent more than 400 ft. below the existing ground water table, while 20 per cent would penetrate material in which the water head would be more than 500 ft. It was hoped, however, that, owing to the fact that a large part of the tunnel would be excavated in hard rock, the quantity of water flowing into the excavations would not be large enough to cause serious difficulties. Hope lost, however, as most of the rock was found to be badly fissured. The only parts of the work which were excavated in comparatively water-free ground were the first 7,000 ft. from the upper portal and the first 450 ft. of the deep shaft. The total inflow reached a maximum of over 20,000 gal. per min., and required the expenditure of 5,600 kw. of electric power for pumping purposes. This amount represented 80 per cent of the total power required for all purposes, including haulage. The handling of this relatively large quantity of water by the type of pumping equipment and pipe lines which could be accommodated in the limited space available in a tunnel of such small size compared to its length in many instances presented problems in the application of pumping equipment which required departure from the ordinary for solution. The problems were often further complicated by the fact that the water usually carried a large load of material of a highly abrasive nature and, in the western end of the tunnel, was heavily charged with highly corrosive carbon dioxide gas, making it very destructive to pumps and piping. In

addition, due to the severe weather conditions which exist during a large part of the year in this high, mountainous area, the continuity of the power supply was often a matter of uncertainty and it was necessary, as far as possible, to locate the more important pumping installations in such relation to the tunnel grade that they would not be submerged during power outages.

Power was supplied to the two portals and the three shafts over a 33,000-volt transmission line from a hydro-power company owning a number of small plants in the vicinity. After transformation it was carried into the work at 2,300 volts by means of rubber-covered cables, and all of the larger pieces of pumping equipment were driven by motors operating at this voltage. Motors of 100 h.p. and smaller were usually operated with 440-volt current, the change in voltage being made by means of portable dry type transformers located where needed along the tunnel line.

For purposes of clarity the pumping equipment used on the project will be classified into the following categories: Camp Service and Fire Pumps, Sinking Pumps, Mop-Up Pumps, and Station Pumps, and the equipment will be treated in that order.

Camp Service and Fire Pumps

As no water suitable for camp purposes was available in the vicinity of one of the portal camps and of one of the shaft camps, it was necessary to lay a 4-inch pipe line some $3\frac{1}{2}$ mi. long, from a large spring to the portal camp, and for an additional 5 mi. from there to the shaft camp. Two motor-driven centrifugal pumps were installed at the spring. Each of these pumps had a capacity of 120 g.p.m. at 480 ft. of head, and they delivered the water into a 100,000-gallon dug reservoir located on a hill adjacent to the portal camp. As the spring was not always accessible in the winter, except by means of snow-shoes or skis, one of the units was equipped with a gasoline-engine auxiliary drive. The engine was connected to the end of the motor shaft by means of an over-running clutch, and was equipped with automatic power failure control.

Because of the nature of the country around the shaft camp it was not feasible to provide adequate storage at this point, nor could the supply line from the portal camp be buried throughout all of its length to a depth which would preclude the possibility of freezing in winter if the flow in the line were allowed to cease. Consequently, a 10,000-gallon steel tank was installed at the shaft camp, and the

pumping installation at the portal camp was laid out with the idea of continuity of delivery at a low rate. The installation consisted of a triplex plunger pump with a capacity of 35 g.p.m. at 1,200 ft. of head for camp service, and a vertical, discharge-pressure-jacketed, split-case centrifugal pump with a capacity of 75 g.p.m. at 1,450 ft. of head for stand-by and fire fighting purposes. The triplex pump was equipped with both electric motor and auxiliary gasoline engine drive, and with a surge alleviator of the spring-loaded type.

The water supply for the other two camps presented no problems worthy of note as there were streams adjacent to both, and it was only necessary to provide pumps with capacity adequate for fire protection, with gasoline-engine stand-by power. The water supply for all camps was chlorinated, however, either by means of vacuum type chlorinators or by means of displacement type hypochlorite feeders.

Sinking Pumps

A test well near the site of the 300-foot shaft indicated that, while the first 200 ft. of the excavation would be in loosely cemented, water-bearing gravel, the remainder would be in tight rock. Consequently, the unwatering of this excavation was accomplished quite simply by the expedient of drilling a 16-inch well to hard rock at a distance of 10 ft. from one side of the location and unwatering the ground by means of a turbine pump. The pump used had a capacity of 1,500 g.p.m., which proved to be large enough to permit the work to be carried on in an almost dry hole.

The unwatering of the excavation for the deep shaft presented a much more serious problem as the test holes in the vicinity had indicated that, while the first 500 ft. would be in hard rock, the remaining 400 ft. would be in material ranging from almost loose sand and gravel to weak sandstone, and that water would be encountered at a depth of approximately 490 ft. Consequently, it did not appear to be likely that a well or wells with sufficient capacity to lower the ground water table could be drilled near enough to the shaft to be effective without danger of the formation of voids which might prove to be a serious menace to the shaft sinking operation. Also, owing to the hardness of the overlying rock, such wells would have been expensive to drill, and would have required pumps with very long columns which themselves would have been very cumbersome to handle and to keep in working condition.

As a relatively large amount of water was anticipated in the shaft sinking operation at this point, two sinking pump units were made available before the water table was reached. Each of these units consisted of a frame designed to run in the guides in one of the two hoisting compartments of the shaft, a sand-settling tank, a vertical 6-stage centrifugal pump with a capacity of 500 g.p.m. driven by a 250-h.p. motor, and a transformer for supplying two or more small pick-up pumps at a safe, low voltage. The pick-up pumps were of the vertical bracket type with a capacity of 250 g.p.m. at 50 ft. of head, and were supposed to deliver the water into the sand-settling tank, which was provided with a blow-off valve and from which the main pump took its suction. The whole "shebang" was supposed to add up to a capacity of 500 g.p.m. at 1,000 ft. of head, was equipped with various gadgets for priming the main pump and supplying clean sealing water to the packing lantern glands at reduced pressure, weighed several tons, was well recommended by all of the experts and specialists available—and the writer—*and was a notable failure.* It was found to be very difficult to regulate the discharge throttle on the large pump so that it would not create a vacuum in the settling tank, which action caused the small pumps to overload their motors. The settling tanks proved to be just tanks; and the large amount of abrasive material in the water, which consisted largely of particles of quartz and obsidian, was fatal to valves, packing sleeves and sealing rings. Also, the shaft bottom was full of pumps, hoses, and electric cables, which were a serious handicap to the miners, and it was necessary to hoist all of this gear into the clear every time a blast was fired.

The failure of these units to perform satisfactorily soon gave rise to the conviction of all concerned that a new line of attack was urgently needed. Consequently, a combination of air-driven, plunger type sinking pumps, lifting the water from the shaft bottom to adequate settling tanks to be located in a pump station in the lower level of the hard rock, was decided upon. The station was located at 485 ft. below the surface, and two suitably baffled, open wooden tanks 5 ft. by 12 ft. by 5 ft. deep, and two horizontal, 4-stage, centrifugal pumps were installed in it. These pumps each had a capacity of 500 g.p.m. at 650 ft. of head, and were so manifolded on the suction side that either pump could be used with either tank to permit cleaning of the other. The arrangement worked quite

satisfactorily while the pumps were new, but the tanks accumulated a load of several yards of material each shift, and required almost continuous mucking out. The centrifugal pumps suffered severely, however, although they were handling much cleaner water than the plunger pumps. This latter statement is well evidenced by the fact that the shrouds of several of the impellers, which were of the single-inlet, back-to-back type, were cut entirely through, opposite the eye, so that the hubs only were left attached to the shaft after a few days of operation. This necessitated the replacement of the pumps with units consisting of two pumps in series with impellers of the double-inlet type, and these proved to be much more able to stand the service.

As the discharge pressure imposed on the plunger pumps became higher with the deepening of the shaft, the service requirements became so severe that the production of new plungers and valves on a quantity basis became necessary, and it became impossible to keep them in operation more than 50 per cent of the time. Consequently, short-coupled, well type turbine pumps were resorted to. The units selected had open bottom impellers and rubber bearings, and were made up with 5 ft. of column between the bowl section and the head. The capacity was 600 g.p.m. at 200 ft. of head, and the units were equipped with heavy strainer bases containing foot valves so that they could rest on the shaft bottom, and with discharge check valves with by-pass holes so that they would be self-priming. By this time most of the working forces on the job had lost faith in any and all innovations, so it was necessary to hang one of these units up in the head frame in a barrel, and to demonstrate that if a bucket of water was poured in the barrel a bucket of water would come out of the discharge nozzle, before sufficient enthusiasm could be aroused to get one of the units lowered down the shaft. The units proved to be very effective, however, and completed the solution of the shaft unwatering problem.

When the bottom of the shaft reached a depth of 709 ft. it was necessary to install another station because of the existence of a large spring at this point. This was done by constructing a concrete collar tank around the shaft. On top of it three pumps, each with a capacity of 650 g.p.m. at 250 ft. of head, were installed. Because of space requirements, these pumps operated at 3,500 r.p.m., and were equipped with double-inlet impellers. It was found that notwithstanding the high operating speed, the units gave very good service.

Mop-Up Pumps

As the tunnel headings progressed, numerous water-bearing fissures were intercepted. Many of these fissures continued to flow large streams with little appreciable diminution throughout the duration of the work. During the early part of the job the flows were handled by means of bracket type centrifugal pumps which were usually suspended in the vertical position in sumps which were excavated at the sides of the tunnel. Usually these were pumps of a capacity of 500 g.p.m. at 100 ft. of head, and were connected to the 14-inch unwatering lines laid on either side of the tunnel by means of hose. Since, at some of the larger springs a number of these units were required, and since they demanded frequent attention, it became evident that pumps of larger capacity would have distinct advantages. As no pumps with the desired capacity, which were not too bulky and too tall to fit into the limited space in the tunnel, were available on the market, and as such good results had been obtained with the turbine pumps used as sinking pumps in the shaft work, it was decided to experiment with equipment of this type for the mop-up work also, by making use of standard turbine impellers and standard vertical motors, and by building up the casings from plate by welding.

The first three units of this type so constructed had a capacity of 2,000 g.p.m. at 50 ft. of head, and, in addition to the impellers, made use of standard diffuser bowl pieces inserted in welded cylindrical casings carrying two 6-inch discharge nozzles and the shaft packing box. Each of these units was driven by a superimposed 50-h.p., 1,750-r.p.m. vertical motor. The units were provided with two discharge nozzles, so that two hose connections not larger than 6-inch could be used, and had heavy strainer bases. The motor shafts were extended to carry the impellers, and no bearings were used in the pump casings. The efficiency of the units was rather low because of the losses in the cylindrical casings. Consequently, nine additional units which were provided with double 180-degree volute casings, were built. These had a capacity of 2,000 g.p.m. at 70 ft. of head, and showed approximately the same performance when tested as the standard pumps for which the impellers had been produced.

As the volume of water being carried by the unwatering lines became greater, the need arose for mop-up pumps which would be capable of discharging into the lines at the higher pressures which existed in the vicinity of the main station pumps. Six additional units, each with a capacity of 1,000 g.p.m. at 160 ft. of head, and two units, each with a capacity of 800 g.p.m. at 200 ft. of head, were built up. These, in turn, had single 360-degree volute casings, and all of them were driven by 50-h.p., 3,500-r.p.m. motors. All, too, were

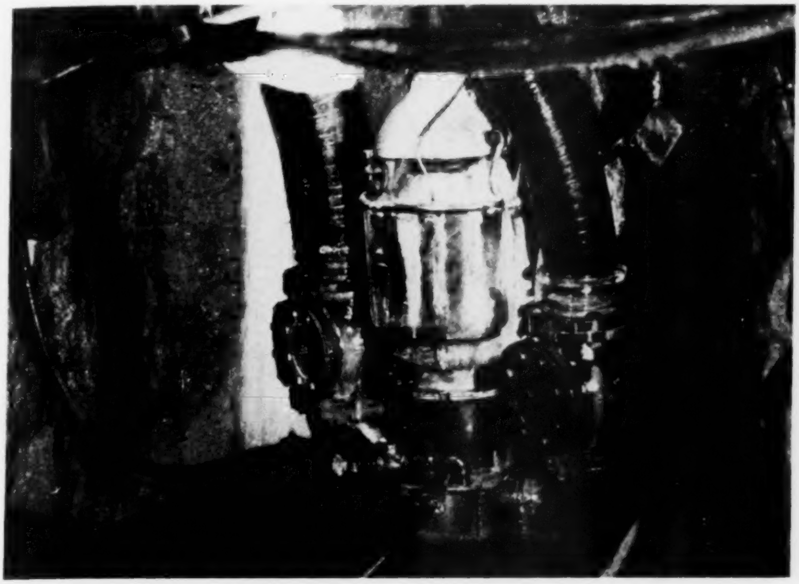

FIG. 1. Built up Mop-Up Pump. Capacity, 2,200 g.p.m. at 63 ft. of head, 50 h.p.

provided with lantern glands in which clean water from the tunnel-drill water line was used and, where gas-charged water was handled, with bronze runner seat rings.

These home-made units (see Fig. 1) proved to be very effective and convenient to handle as they could be set down in any small sump on their wide strainer bases, and required only the connecting of the discharge hoses and the power supply to make them ready for operation.

Station Pumps

Main pumping stations were cut in series with the unwatering lines at Tunnel Stations 7800, 11000 and 15000, and a station was installed at the foot of each of the three shafts. The aggregate capacity of the pumps installed in these five stations was 62,000 g.p.m., and of the motors, 9,450 h.p. This relatively large capacity of equipment, as compared with the 20,000 g.p.m. maximum amount of water to be handled, was necessary to provide for sudden blow-ins,

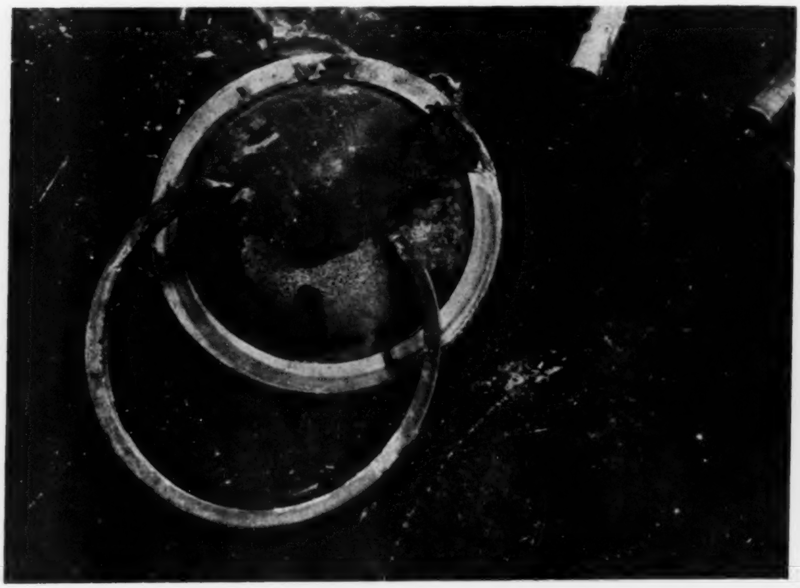

FIG. 2. Disc and Seat Ring from a check valve of 400 lb. per sq. in. working pressure, showing erosion by sediment in water after 6 months service

and to permit units to be taken out of service for overhauling as, owing to the large amount of abrasive and corrosive material in the water, the service life of such parts as wear rings, shaft sleeves and impellers (see Figs. 2 and 3) was often rather short. All of these station pumps were driven by 2,300-volt motors.

All of the stations along the line of the tunnel, and also the station at the ventilating shaft, were located in the section of the tunnel beginning with the high portal. Consequently, it was necessary to

place the equipment at a sufficient height above portal-invert grade to prevent submergence in the event of pipe line or power failure, both of which occurred a number of times during the course of the work. This necessitated the elevation of some of the stations remote from the portal to a height too great to permit the raising of the water from the tunnel level by suction lift, and made necessary the use of well type turbine pumps in series with the line pumps for this purpose.

FIG. 3. Cast-Iron Impellers After 2 Months Service in Station Pumps, showing corrosion by CO_2

A representative station of this kind was the one at Tunnel Station 15000. The floor there was elevated 22 ft. above the floor of the tunnel to insure that the water would flow out at the portal without reaching the level of the equipment, even though there might be several stalled trains and other obstructions to flow in the tunnel. This station contained two horizontal pumps, each with a capacity of 4,500 g.p.m. at 210 ft. of head, driven by 300-h.p. motors, and two turbine pumps, each with a capacity of 5,000 g.p.m. at 60 ft. of head, for raising the water from a sump in the tunnel floor. The line pumps were so manifolded that they could take their supply from

either or both unwatering lines or from the turbine pumps, and so that they could deliver into either or both unwatering lines.

The wisdom of these rather elaborate precautions against flooding was amply demonstrated rather late in the job when a power outage of approximately five hours duration occurred. In this instance the tunnel was flooded to within 7,800 ft. of the high portal before the power supply was restored, but the tunnel was unwatered and work was resumed in approximately ten hours after restoration, and the only motors which required drying out were the 440-volt motors on the mop-up pumps.

The station at the foot of the 300-foot shaft was in hard rock. Consequently, it was possible to excavate a pump room with a large sump for sediment-settling purposes beneath it and adjacent to the shaft. This was an ideal arrangement as the material could be blown directly into one of the skips for hoisting to the surface. The equipment in this station consisted of two 2-stage pumps, each with a capacity of 2,500 g.p.m. at 380 ft. of head, and two single-stage pumps in series with a capacity of 4,500 g.p.m. at 210 ft. of head. All of these pumps were driven by 300-h.p. motors. This station gave very little trouble as the sump was very effective in removing the sediment from the water, but the station was flooded to a depth of 200 ft. on one occasion when a sudden blow-in of water occurred. This occurred before either of the headings in this section had been holed through, and necessitated the unwatering of the work by means of two 2,500-g.p.m. capacity well type turbine pumps installed in the shaft and the hoisting of all electrical equipment to the surface for drying out.

The foot of the 900-foot shaft was located in very weak sandstone. Consequently, it was not feasible to excavate an adequate pump room or sedimentation sump, and it was necessary to tuck the five units installed at this point in two small drifts and along the sides of the tunnel on one side of the foot of the shaft. These units were of the 4-stage, single-casing type with a capacity of 2,500 g.p.m. at 950 ft. of head each, and each was driven by two 400-h.p. motors in tandem. This selection of motors was made because of the small size of the space available for installation, and to avoid the necessity for bulky, reduced-voltage starting equipment. The two motors on each unit were started consecutively by means of push-button-operated, contactor type full-voltage starters, thus producing a smaller line voltage disturbance than would have been the case if single 800-h.p. motors

and reduced-voltage starters had been used. As it was necessary to regulate the output of these pumps by throttling, each unit was equipped with an angle type throttle valve and a gate type stop valve. These angle valves lasted throughout the job although they were subjected to very hard service. The pumps themselves did not fare so well, however, and it was usually necessary to replace the complete rotating element and the wear rings and inter-stage bushings after every thirty days of operation. The units discharged to the surface through two rising columns, 8-inch and 16-inch respectively.

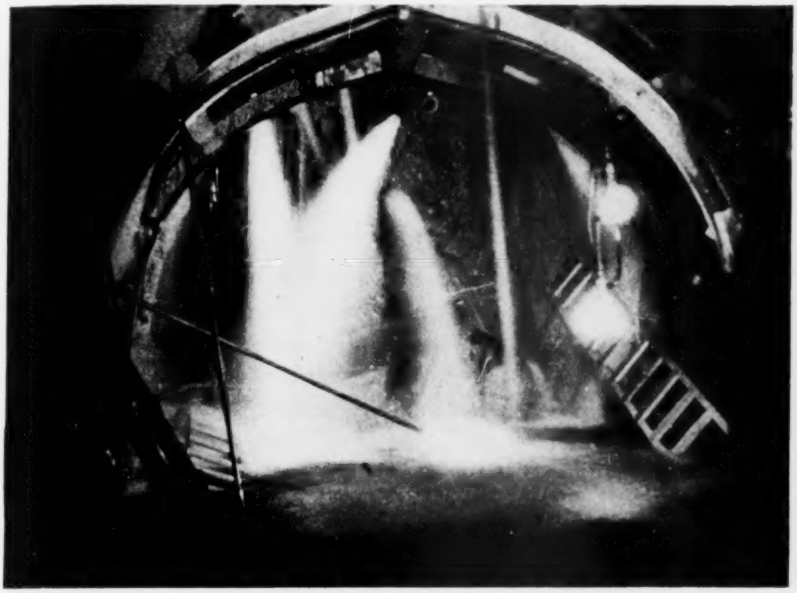

FIG. 4. Tunnel Station, showing approximately 1,500 g.p.m. of water, highly charged with CO_2, coming from test and pilot holes in face

Hope and luck were relied upon to maintain the continuity of the power supply until the heading from the deep shaft toward the high portal was holed through. They were successful. As a safeguard, however, a concrete bulk-head with a heavy, water-tight steel door was built across the tunnel at a short distance from each side of the shaft to protect the equipment from immediate flooding in the event of power failure, and a mop-up pump with a capacity of 8,000 g.p.m. at 25 ft. of head was built up and installed to discharge the water, which found its way into the pump room, through the bulkhead into

the section of the tunnel toward the high portal. This pump was driven by a 75-h.p. motor, and was similar to the smaller home-made mop-up pumps in design. Power was supplied to it from a 250-h.p. gasoline-engine-generator set at the surface. This arrangement proved to be a life saver during the long power outage previously mentioned, as the flooding of the pump station would have entailed a very expensive unwatering operation in the shaft and the hoisting of all equipment to the surface for drying.

As the foundations for the rising columns and other equipment left little space in the shaft pocket, this space was unwatered by means of two pumps of the submersible type. Each of them had a capacity of 500 g.p.m., and was installed in a vacuum chamber equipped with a foot valve so that it would be self-priming without submergence. This setup was satisfactory after it was discovered that the vacuum upset the mercury seal and a suitable vent was provided.

Removal of Muck

Another problem which presented itself at this point, and which was closely related to the pumping problems, was the removal of the muck which was spilled into the shaft pocket by the gates in the muck hoppers. This pocket was approximately 50 ft. deep, and the accumulation of this material made it necessary to discontinue hoisting operations periodically so that it could be hoisted to the tunnel level. There was space at the bottom of the shaft for the installation of a hopper to receive this material but no space adequate for any sort of conveyor to raise it. The problem brought to mind the device which is used on ship board to discharge ashes overside, and, as plenty of water at high pressure was available, a large ejector, capable of passing 4-inch rocks, was built for returning the material to the tunnel level. The scheme worked perfectly.

The pump room at the foot of the ventilating shaft was located at a height of 18 ft. above tunnel grade, and contained two 4-stage, single-casing units, each with a capacity of 5,000 g.p.m. at 500 ft. of head and driven by 800-h.p. motors. These units had been purchased early in the job for use at the 900-foot shaft, were of the 4-stage, single-casing type with an external connection between the second and third stages, and were designed for a capacity of 2,500 g.p.m. at 1,000 ft. of head. As they were on hand when the ventilating shaft became necessary they were installed at this point and

converted to double 2-stage units by removing the inter-stage connection and connecting two suctions and two discharges to each unit. As these units were often operated at less than full capacity, it was necessary to use considerable care in so regulating the two throttle valves on each unit that both pairs of stages had approximately the same output to avoid the development of an excessive load on the thrust bearings. After some practice, however, the operators learned to do so without trouble, and as an adequate sump had been excavated beneath the pump room to rid the water of sediment, very little trouble was experienced with them. Each of these pumps was served with water from the tunnel level by a turbine pump with a capacity of 5,000 g.p.m. at 120 ft. of head, driven by a 200-h.p. motor.

Maintenance and Operation

Owing to the highly abrasive nature of the material carried by the water, the maintenance of pumping equipment was a serious problem. Such spare parts as sealing rings and shaft sleeves were made of mild steel and were deeply case hardened. The metal spray process was used to a large extent also. Where hardness was required in the built-up surface, spring steel wire was used, the material so produced requiring finishing by grinding. Stainless steel and Monel metal were also used to a large extent, and worn bronze parts were often built up with these materials, Monel metal being preferred as it showed less tendency to crack and loosen on cooling. Impellers, when worn, were built up with bronze welding rod. Casings, when not worn beyond repair, were built up with nickel iron welding electrode and refinished to size. As serious wear usually took place only in the sealing ring seats, re-machining after welding was simple.

Impellers of both high strength cast iron and bronze were used, but the latter material was found to be the better, even where corrosion was not a controlling factor, as the iron impellers often developed cracks after short periods of operation. It was not unusual to find one or more impellers in some of the larger units completely separated from their hubs when the units were shut down for repairs. In the multi-stage units having single-inlet impellers it was also found to be essential that the joint between the bore of the impeller and the shaft and keyway was absolutely water-tight, as the sediment washing through any small interstice would soon enlarge it until the im-

peller would be loose, or even free to turn on the shaft. This was particularly true in the large pumps in the station at the foot of the deep shaft.

Many types of shaft packing were tried on the job. Cheap, graphited hemp proved to be the most economical where pressures were low. For the high pressure pumps unusually good service was obtained from a packing composed of a mixture of lead and wood particles impregnated with graphite and enclosed in cotton tubing.

When the large inflow of gas in the volcanic zone between Tunnel Stations 9000 and 19000 was encountered the problem of ventilating the tunnel became acute, and the single 4,000-cu. ft. per. min. blower discharging through an 18-inch ventilating line for this heading was entirely inadequate. The inflow of gas was continuous, and reached a maximum of 1,500 cu. ft. per min. The problem was further complicated by the fact that the supply of electric power to the project was subject to interruptions of considerable duration, and a cessation of the air supply for even a few minutes caused the concentration of gas in the tunnel atmosphere to increase to a point which rendered breathing impossible. Under these conditions dependence on luck and quick action for getting the working forces out of a dark tunnel over three miles of very bad railway was a very risky business. Consequently, the ventilating shaft was sunk to tunnel grade and four blowers, each with a capacity of 10,000 cu.ft. per min., were installed. These blowers were of the centrifugal type and were directly driven by 100-h.p., 3,500 r.p.m. motors. In addition, to provide for operation during power outages, two of these blowers were arranged for operation by 16-cylinder, V-type automobile engines. This was done by extending the shafts of the electric motors and coupling the engines by means of simple sliding-jaw couplings. The starting of these engines was made a routine operation at the beginning of each shift to insure their being in working order at all times. The blowers were connected by means of a system of manifolding to three 18-inch ventilating lines which extended down the shaft and to the working face. Different methods of operating these lines were tried out, such as operating one of them as an exhaust line and the other two as blowing lines; and the operation of all three as blowing lines. The latter method was the one finally adopted as it provided the best atmosphere at the working face.

In order to prevent the gas-laden air from the gas zone from travel-

ling back through the section of the tunnel between the ventilating shaft and the high portal, an air-lock with air-operated balancing gates and hydraulically operated main doors was installed. The distance between the air-lock bulkheads was 80 ft., or enough to permit the handling of a locomotive and six tunnel cars. The control consisted of a switch at each end which energized the magnetically-operated air valves on the balancing gates. When these gates were wide open they tripped the valves on hydraulic cylinders

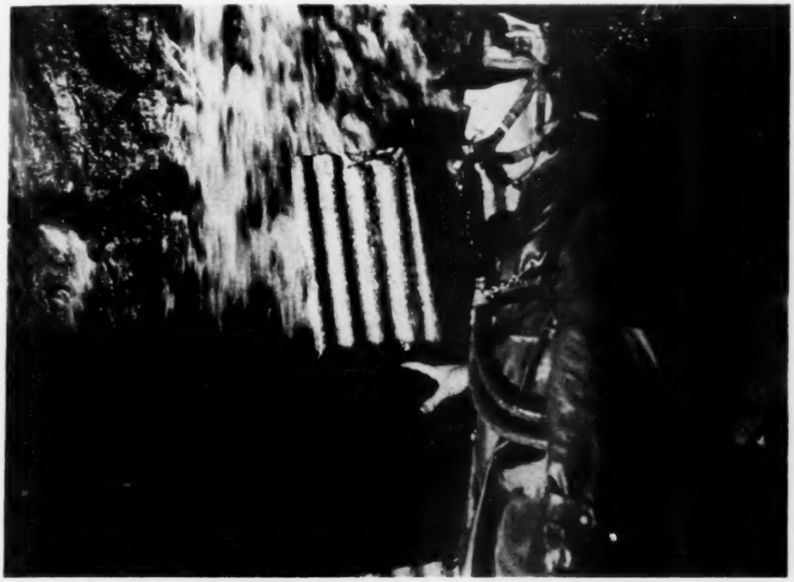

FIG. 5. Mine Rescue Man, fully equipped, standing next to a cascade of water high in CO_2 content

which operated the main doors. For ventilating the section of the tunnel between the air-lock and the high portal, a large-capacity, low-pressure blower with electric motor and auxiliary engine drive was installed at the collar of the ventilating shaft.

An efficient mine rescue squad was organized, and was fully equipped with oxygen helmets and a mine car carrying oxygen tanks, spare helmets and resuscitating apparatus (see Fig. 5). This squad was never called upon to do any actual rescue work, but on one occasion, when the ventilating plant was damaged badly by a snowslide, they

were called upon to take over the operation of the pumping equipment for two days, and succeeded in preventing the flooding of the tunnel until the air supply was restored.

The work of driving the tunnel was done under the general supervision of H. A. Van Norman, Chief Engineer and General Manager of the Bureau of Water Works and Supply of the Department of Water and Power, and under the direct supervision of H. L. Jacques, Engineer of Major Construction of the Bureau. The installation, maintenance, and operation of the above equipment was performed under the supervision of W. W. Wallace, who had charge of all mechanical and electrical work on the job.

Novel Design Features of the Lansing Water Conditioning Plant

By Claud R. Erickson

THE City of Lansing obtains its water from 70 wells drilled into the Saginaw formation, an alternating series of sandstone and shale underlying the city. The wells are mostly 12 to 14 in. in diameter; they are cased through about 80 ft. of gravel and clay and are drilled into the rock an additional 300 to 400 ft. These wells are located in various parts of the city and furnish the water which is pumped by both air lift and deep well turbine pumps to the water conditioning plant. The water has a uniform temperature of 51°F. as it comes from the wells. Its total hardness varies from 313 to 515 parts per million with an average of 464.

The people of Lansing have always been proud of their water supply. This has been due to its good taste, cool temperature and freedom from dangerous contamination. The water in the past has, however, had several undesirable features in that it was hard, scale forming, often red due to its corrosive properties, and in that it contained considerable iron and iron bacteria. Table 1 shows a typical analysis of the raw water. It was to correct these undesirable features that the water conditioning plant was constructed and placed in operation in the latter part of December, 1939.

The first concrete move toward conditioning the water supply was made in 1922 when Alvord, Burdick and Howson, Engineering Consultants of Chicago, were employed to report on its possibilities. Then, in 1923 an experimental plant was built to conform with their recommendations.

The present site of the plant was selected because it was near the center of the water distribution system, because water from most of

A paper presented on April 23, 1940, at the Kansas City Convention by Claud R. Erickson, Mechanical Engineer, Board of Water & Electric Light Commissioners, Lansing, Mich.

the wells could easily be brought there, because the existing 7,000,000-gallon ground-level reservoirs could be used for clear wells, and because the pumping station is located on adjacent property.

Conditioned water flows by gravity to the clear well reservoirs from where it is pumped by the Cedar Street Pumping Station into the distribution piping under a pressure of 65 lb. The average pumpage to the water distribution system is 9 m.g.d.; during maximum days 17 m.g.d. are required. For periods of a few hours during the summer, water is sometimes used at the rate of 30 m.g.d. The present wells are capable of supplying 20 m.g.d.; the water conditioning plant is designed for a maximum flow of 30 m.g.d. with provision for future extensions if necessary. The population served by the plant is 87,000. The distribution system has approximately 200 miles of mains and 20,000 services.

TABLE 1

Typical Analysis of Raw Water

	p.p.m.
Calcium	125.1
Magnesium	37.3
Sodium	7.0
Total Metallic Ions	169.4
Bicarbonate	394.6
Sulfate	123.7
Chloride	19.1
Total Non-Metallic Ions	537.4
Total Iron	1.0
Total Solids	538.4
Free Carbon Dioxide	40.0
Dissolved Oxygen	6.0
Alkalinity (as Calcium Carbonate)	323.0
Non-Carbonate Hardness (as Calcium Carbonate)	141.3
Total Hardness	464.3

pH:	7.2
Temperature:	51°F.
Color:	Clear
Turbidity:	10 p.p.m.

Early in 1938, when the City of Lansing did not have sufficient

projects to put available men on W. P. A. rolls to work, it was decided to proceed with the construction of the plant. Alvord, Burdick and Howson were again engaged as engineering consultants and Lee Black and K. C. Black of Lansing as architectural consultants. C. P. Hoover, of Columbus, Ohio, was consultant on the chemical process. Detail plans and design work for architectural, structural, mechanical, electrical, and heating and ventilating features were accomplished by the Engineering Staff of the Board of Water and Electric Light Commissioners under the supervision of Otto E. Eckert, General Manager, and the author. Functional plans upon which the general design was based were prepared by the engineering consultants. In turn all plans prepared by the board were reviewed and approved by the consultants prior to construction. This policy indicates the close cooperation of the staff on a project where excavation and plans were started almost simultaneously. Because the work was to be done largely with W. P. A. labor, architectural concrete was selected as the best type of material to make use of the largest number of unskilled laborers and to provide a structure free from rust and decay.

Functional Design.

The plant is in reality two units in parallel, with each half having a maximum rated capacity of 15 m.g.d. Equalization or interchange gates between the first carbonation basin effluents, the second settling basin effluents, and the filter effluents, together with control and separate metering of the common raw water supply, provide additional flexibility to the plant either for conditioning treatment or periodic maintenance cleaning.

Figure 1 is a flow diagram of the water supply system. The time noted in parenthesis is the retention period for a flow rate of 20 m.g.d., the total of which is about 6 hr. for the entire plant. Piping, rate of flow controllers, etc., have all been designed for the maximum of 30 m.g.d. Flumes are large enough so plant capacity may be tripled and still be served by the existing head house. The head house is located in the center and is flanked on the north by the mixing and coagulation section and on the south by the filter section. Future expansion will only duplicate these flanking units. The head house provides for chemical handling, wash water storage, laboratory, lavatory and locker room, janitor and maintenance space, pump room for plant services, filter wash water reclaim basin, local air-lift well-field basin for booster pumps, and lobby space.

The mixing and coagulating section provides for aeration, chemical storage, chemical feed, rapid mix, flocculation, first settling and carbonation. The filter section provides for second settling—containing a recarbonation grid, filters and pipe gallery—and final recarbonation.

Architectural Design

The building, of monolithic architectural concrete, is of modern design. The exterior treatment is rendered in natural concrete,

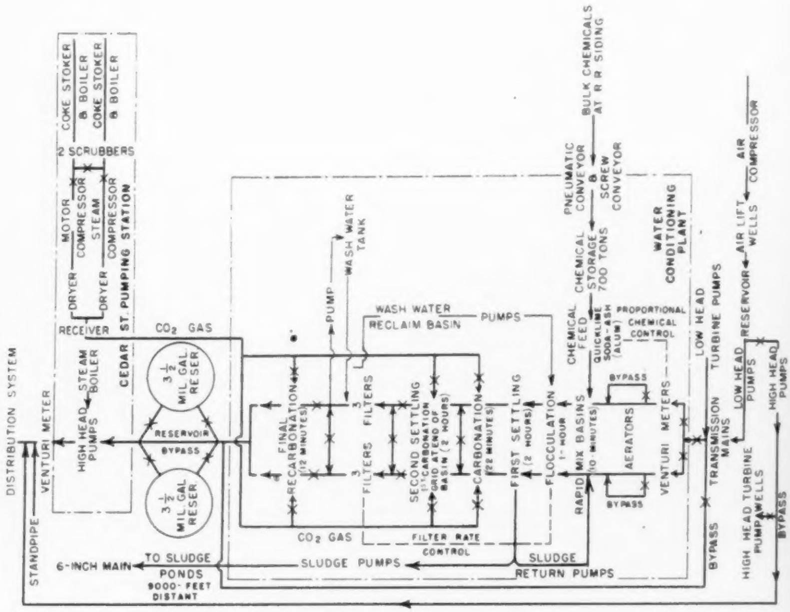

FIG. 1. Flow Diagram of Water Supply System

aluminum, and glass block, with the exception of the main entrance of bronze and the hip roofs of batten seam copper. As the water carrying basins extend 18 ft. above the street grade, window requirements above this level controlled the general design.

On the east facade of the head house is a 32-foot figure symbolic of furnishing conditioned water to the populace. The conventional symbol of water is exemplified by the wave pattern on the coping of the head house and chemical storage section. To break up the remaining wall surfaces and to give scale to the building, geometric

ornament is used together with rectangular scoring, the horizontal score predominating.

For better insulation, glass block is used in the window openings. The blocks are set back in the wall to permit an inward slope of the sills so that window drainage can be collected by a conduit system cast in the structure to prevent its staining the face of the building. The filter gallery is lighted by a continuous glass block clearstory. As the entire plant is roofed, the proper circulation of air in the

FIG. 2. New Water Conditioning Plant

aeration basin is obtained by the substitution of wide band aluminum louvres, backed by screens, for the glass block as used in the window openings.

With the exception of the entrance and lobby, the entire interior finish is exposed concrete construction with aluminum trim. Terrazzo flooring is employed only on entrance, entrance stair, and operating floor of the head house, filter gallery, and chemical feeder room. The entrance and lobby, two stories high, have "Rostone" artificial stone-wall facing with a plaster ceiling lighted with concealed

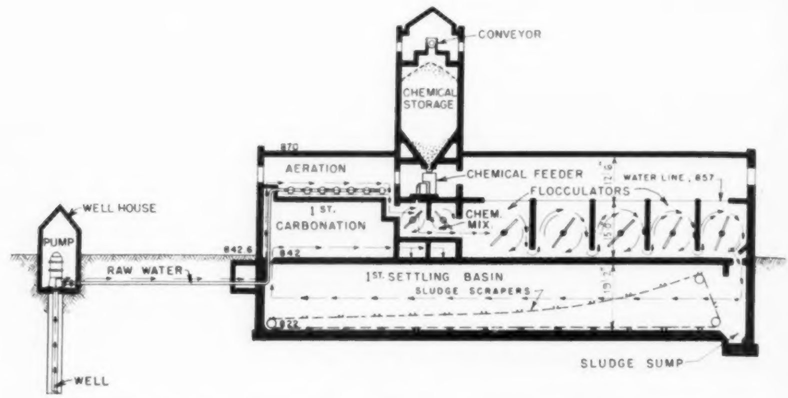

FIG. 3. Mixing and Flocculating Basins, Settling Basin and Chemical Storage Section

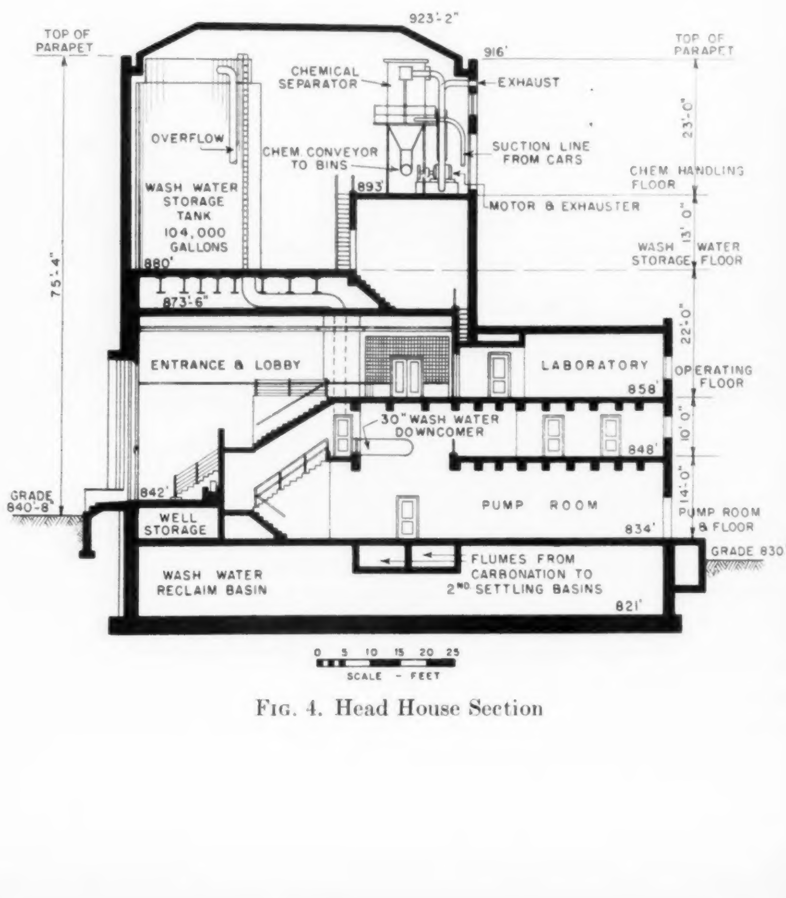

FIG. 4. Head House Section

aluminum cove lights. The entrance is centered by a fountain of modern design consisting of blue-glazed terra cotta statuary. Modernistic aluminum rail flanks the stairways which run from each side of the entrance, converge in the center and lead up to the second, or operating, floor. Above the wainscot the walls of the lobby at this level are covered with three large mural paintings which portray the production of utilities by the Board of Water and Electric Light Commission and the constructive and destructive forces of water.

The landscaping carries out the horizontal lines of the building with a juniper hedge, hawthornes serving as an accent at the entrance and

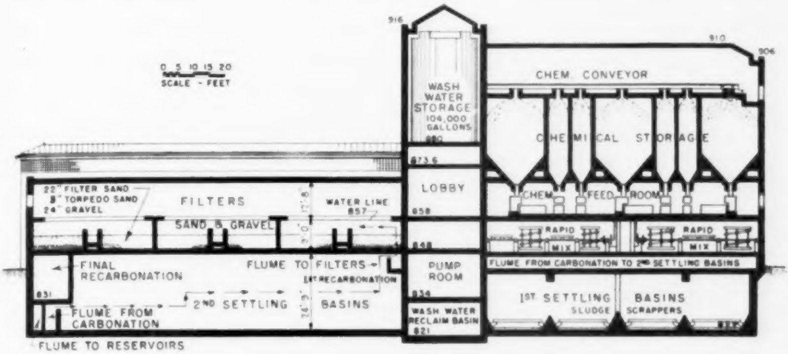

FIG. 5. Longitudinal Section

corners. Reflectors in the street lights, at the entrance, floodlight the head house area for night lighting.

Structural Design

Suitable soil for the foundation was not available above an elevation of 20 ft. below the street grade. To make use of the present reservoirs without repumping, it was necessary to establish the general operating floor level 9 ft. 6 in. above the reservoir overflow or 18 ft. above the street grade. With so much depth available, the basins were double decked—first and second settling basins under the flocculators and filters respectively. This condition required a design for a hydraulic head of 35 ft. throughout the plant. As the structure below the operating floor is essentially water-containing basins which will be held at the constant temperature of the water, it was designed as one monolithic structure without expansion joints,

even though the over-all dimensions are 140 ft. by 240 ft. 6 in. While this procedure eliminated problems of watertight construction at expansion joints, it necessitated care in design for uniform soil pressures to guard against unequal settlement, if any. Accordingly the total weight of the structure (maximum of 62,000 tons) is spread over the entire foundation area by a mat footing. This design gave soil pressures ranging from 3,300 lb. to a maximum of 5,500 lb. per sq. ft., even under conditions of eccentric loading. The foundation soil was judged to be capable of withstanding loads of from 10,000 to 16,000 lb. per sq. ft.

Above the operating floor, expansion joints were provided separating the head house section from the mixing and coagulation section on one side, and the filter section on the other. The mixing and

FIG. 6. Transverse Section; Settling and Filter Basins

coagulation section in turn has expansion joints isolating the high chemical storage portion. Each unit above the operating floor, tied into and including the entire structure below the operating floor as a single unit, was designed so that all members would resist, at all sections, the maximum bending moments and shears produced by dead load, live load, and wind load, as determined by the principle of continuity. In the main, Professor Hardy Cross's method of moment distribution was employed for this rigid frame analysis.

Limiting specifications were the requirements as set forth in "Building Regulations for Reinforced Concrete" of the *American Concrete Institute Building Code* (*A. C. I. 501-36-T*). For design purposes for concrete in contact with the water, the concrete was considered as having an ultimate compressive strength (f'_c) of

2000 lb. and for concrete above water 2,500 lb. per sq. in. Maximum unit stress for reinforcing steel was 18,000 lb. per sq. in. with 2-inch concrete protection and 20,000 lb. per sq. in. with $1\frac{1}{2}$-inch concrete protection respectively, for the two cases above. The lower unit stresses were employed for the water-containing basins to compensate for the reduced strength of wet concrete (see Table 2).

Due to the fact that the magnitude of the structure, without expansion joints, caused severe requirements for shrinkage and temperature reinforcement, and because a structure as free from cracks as possible was desired, this reinforcement was provided in the ratio of reinforcement area to concrete area of 0.004. This ratio was applied for walls and slabs up to 18 in. in thickness and the same maximum value ratio was used for all thicker walls and slabs. Temperature and shrinkage reinforcement was increased at all corners and abutting sections to provide full continuity of the main body section of the reinforcement.

Most of the structure employs the solid slab, beam, and girder construction for the frame, the principle exception being the second settling basin floor and its ceiling which supports the filters. The latter is designed using the two-way flat slab with dropped panels to minimize obstructions which might cause turbulence in the water. The ceiling slab has a maximum possible downward loading of 1,050 lb. per sq. ft. and an upward maximum loading of 360 lb. per sq. ft., depending upon the condition of the filter and settling basin. A comparison of these figures with the usual building loading of 125 lb. per sq. ft. partially explains the high reinforcing steel factor for the structure—230 lb. per cu. yd. of concrete. All exterior walls of the structure were designed for full hydraulic loading without support from the backfill, and likewise for full backfill loading with the basins empty.

The floor supporting the wash water tank in the upper part of the head house has a maximum load of 1,600 lb. per sq. ft. As this load is applied and removed with comparative rapidity, its design is very similar to a steel-deck girder bridge—structural steel girder sections, with bearings on an encased structural section, supporting a deck slab of concrete. The floor slab and the structural girders are entirely isolated from the rest of the structure by expansion joint material, leaving the floor free to deflect with the load (the girder span being computed as a simple span) without influencing the rest of the structure.

TABLE 2

Unit Stresses and Design Factors

DESCRIPTION	SYMBOL	ALLOWABLE UNIT STRESS	
		Concrete in Contact With Water	Concrete Above Water
		lb./sq. in.	
Ultimate compressive strength of concrete at 28 days	f'_c	2,000	2,500
Ratio of modulus of elasticity of steel to that of concrete	n	15	12
Flexure:			
Extreme fiber stress in compression	f_c	800	1,000
The same adjacent to supports of continuous or fixed beams or of rigid frames	f_c	900	1,125
Shear:			
Beams with no web reinforcement and without special anchorage of reinforcing steel	v_c	40	50
The same with special anchorage	v_c	60	75
Beams with web reinforcement without special anchorage of longitudinal reinforcing steel	v	120	150
The same with special anchorage	v	240	300
Flat slabs at a distance *d* from edge of column capital or dropped panel	v_c	60	75
Footings without special anchorage of reinforcement	v_c	40	50
The same with special anchorage	v_c	60	75
Bond:			
In beams and slabs and one-way footings, deformed bars without special anchorage	u	100	125
The same with special anchorage	u	150	180
Two-way footings, deformed bars without special anchorage	u	75	94
The same with special anchorage	u	110	140
Bearing:			
On full area	f_c	500	625
On one-third area	f_c	750	938
Tension in reinforcing steel	f_s	18,000	20,000

TABLE 2—*Concluded*

DESCRIPTION	SYMBOL	ALLOWABLE UNIT STRESS	
		Concrete in Contact With Water	Concrete Above Water
		Design Constants	
Ratio of effective area of tensile reinforcement to effective area of concrete	p	0.0089	0.0094
Ratio of depth of neutral axis to depth from compression surface of beam or slab to center of longitudinal tensile reinforcement	k	0.4000	0.3750
Ratio of lever arm of resisting couple to depth *d*	j	0.8667	0.8750
$\frac{1}{2}f_ckj$ or pf_sj	K	138.7	164.1

The three large chemical storage bins were designed for pebble lime, with four smaller bins designed for either soda ash or alum, depending upon which material would produce greatest stress. This provided for interchange in storage of materials in the smaller bins. While the original design took into account both the vertical and lateral pressures as determined by: (1) Jansen's formulas for pressures in deep bins; (2) Coulomb's wedge theory; (3) Link-Belt Company's tabular values for surcharged retaining walls as applied to the lime bins; and (4) a 75 per cent fluid pressure for soda ash as recommended by the Fuller Co.; and provided for the maximum stress thereby, a review of the design indicated that stresses determined by the following equivalent fluid pressures would produce a comparable design. Other factors assumed are shown in Table 3.

The bin bottoms were designed for the component of the total vertical load and the lateral pressure for the corresponding depth. To minimize cracking in these rectangular bins, all corners were beveled and provided with negative reinforcement whether statically necessary or not. High carbon steel was specified for the lateral reinforcing of the bin walls. Bin bottom slopes were: for the lime bins—sides, 54.8° and 50.2° and corners 42.6°; and for the soda ash and alum bins—sides, 54.8° and 60.0° and corners, 47.6°.

Air-Conditioning

Due to the large operating space required by the equipment, the tempering effect of the water, and the dust-free operation of the chemical handling and feeding equipment, air-conditioning as applied to this plant resolves itself into only two functions: condensation control and an emergency exhaust system. Carbon dioxide is generated and steam for heating is produced outside the plant at the pumping station some 600 ft. distant, thus removing any need for air-conditioning occasioned by boiler operation.

Condensation occurs when surfaces are below the dew point or saturation temperature of the water mixed with air. As there is a large volume of water continually flowing through the plant at the well temperature of 51°F., all open space tends to be saturated continually with water vapor at the temperature of the water. Elimination of condensation within the plant during the winter requires merely that surface temperatures be kept above the well temperature of the water.

TABLE 3

Chemical Bin Design Factors

SYMBOL	DESCRIPTION	LIME	SODA ASH	ALUM
w	Weight per cu. ft., *lb.*	55	45	65
l	Equivalent fluid pressure-lateral pressure, *lb. per sq. ft.*	25	10	18
ϕ	Angle of repose, *degrees*	35	40	30
u′	Coefficient of friction on bin walls	0.70	0.84	0.58
k	Ratio of lateral pressure over vertical pressure, Janssen's Formula	0.4	0.6	0.4

The inside surface temperature of walls is determined by a percentage or ratio, *R*, of the total difference in temperature between inside and outside air, or

$$R = \frac{\text{inside temp.} - \text{temp. inside surface}}{\text{inside temp.} - \text{temp. outside air}}$$

This ratio is constant for any particular kind of wall and wind condition and for all temperatures of outside air. It is based on the fundamental principle that the drop in temperature through a wall

is proportional to the resistance to the transfer of heat. The value of R is then equal to the ratio of the inside surface resistance to the total resistance to transfer of heat, or

$$R = \frac{\text{overall heat transmission coefficient}}{\text{inside film coefficient}}$$

From these two ratios the inside surface temperature may be calculated for any particular wall with various inside and outside temperatures. Table 4 illustrates the results of such calculations. From the table it will be seen that so long as the inside temperature is maintained at 70°F. and outside temperatures do not get below 0°F., inside surface temperatures will be above 51°F., as compared

TABLE 4

*Inside Surface Temperatures—Winter**

OUTSIDE TEMPERATURE	INSIDE SURFACE TEMPERATURE		
	Window Sash	4" Glass Block	14" Concrete Wall
0°F.	17.5°F.	56.5°F.	51.5°F.
10	25.0	58.4	54.1
20	32.5	60.4	56.2
30	40.0	62.3	59.4
40	47.5	64.2	62.0
50	55.0	66.1	64.7
60	62.5	68.0	67.3
70	70.0	70 0	70.0

* Inside Temperature—70°F. Air velocity zero. Outside wind velocity—15 miles per hour.

to a minimum of 45°F. outdoor temperature for the window sash. No condensation will occur in the plant as long as inside surface temperatures are above 51°F. This factor dictated the use of glass block for window openings.

Sufficient radiation has been placed in the plant so that an inside temperature of 70°F. can be maintained. No attempt is made to heat the flocculating basin as condensation is not objectionable in the room. The chemical handling room is heated to prevent the accumulation of any condensation which might possibly percolate into the chemical storage bins. In the chemical feed room the heating element, being placed at one end of the long room, lacked sufficient air velocity to lessen the resistance of the air film on the opposite

and exterior wall. Propeller type electric fans are now mounted to force the air against this wall, reducing the air film sufficiently to bring the surface temperature above 51°F.

While the plant has not yet been operated through the summer months, the only interior walls likely to be below the saturation temperature of the water are in the pipe gallery, pump room, and lobby entrance. In these rooms the saturation temperature and the amount of moisture will depend upon the amount of infiltration of outside air. Protection for the lobby walls has been secured by a 2-inch hollow tile separation between the water-containing basins and the "Rostone" surfacing and this should maintain a high enough surface temperature to be free from condensation.

The pump room and pipe gallery are isolated by weather-stripped doors so that condensation may be eliminated in them by lowering

TABLE 5

*Outside Surface Temperatures—Summer**

OUTSIDE TEMPERATURE (DRY BULB)	OUTSIDE SURFACE TEMPERATURE 14" CONCRETE WALL
70°F.	64.3°F.
65	60.8
60	57.3
55	53.8
51	51.0

* Inside temperature 50°F. Wind velocity—zero.

the humidity. The humidity control is effected by a chemical dehydrator manufactured by H. J. Kaufman, of Detroit. The unit holds a charge of 140 lb. of calcium chloride with a circulating air fan of 500 cu. ft. per min., giving a rated capacity of about 5 lb. per hr. of moisture removal. It is expected that the use of this dehydrator in the pipe gallery will eliminate the nuisance of dripping condensation during the summer months.

Condensation on outside wall surfaces below the water line will occur only when the saturation temperature or dew point is above the temperature of the outside wall surface. Temperatures of outside wall surfaces are calculated in the same manner as described above for inside surfaces (see Table 5).

Those days in which the dew point is above the temperature noted for outside surfaces with the corresponding dry bulb temperature,

as indicated by weather reports, are very infrequent, with the exception, of course, of rainy days.

The emergency exhaust system consists of a two-speed exhaust fan, rated at 13,500 cu. ft. per min., with ducts leading to the several rooms of the plant to aid in ventilation in case of failure in the proper functioning of either chemical handling or chemical feeding equipment, or to provide for the emergency of escape of carbon dioxide gas from the carbonation basins or piping.

To prevent increase in the temperature of the water during its retention in the plant, all roofs have been insulated with one inch of cork. Also flat roofs suitable for flooding, if desirable, are provided over the open water basins. Calculations indicate that there will

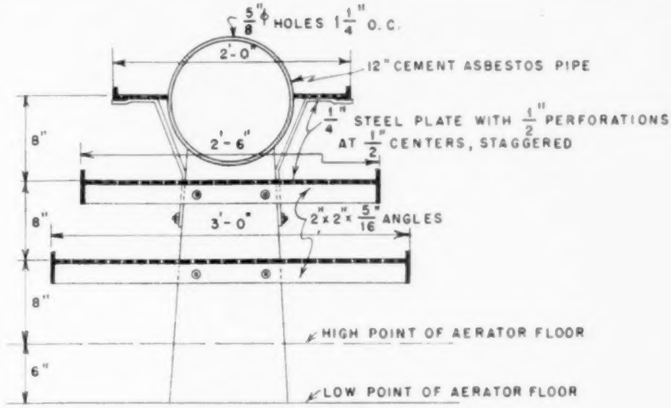

FIG. 7. Typical Aerator Section

be less than a half-degree temperature rise through the plant at a 10-m.g.d. rate, with a lesser rise, of course, for higher rates.

Aerators

Each aerator consists of a header flume with 8 stop-gate openings leading into 12-inch cement-asbestos pipes that are 27 to 33 ft. long and are drilled with $\frac{5}{8}$-inch openings on $1\frac{1}{4}$-inch centers along the top of the pipe. From these openings the water cascades over the pipe through 3 perforated trays placed at 8-inch centers below the top of the pipe. Each succeeding tray is wider than the one above to permit cascading over the edge in case all perforations become clogged between cleaning periods. The trays are $\frac{1}{4}$-inch steel plate with $\frac{7}{32}$-inch holes on $\frac{1}{2}$-inch staggered centers (see Fig. 7). The floor of

the aerator room has a pitch of 6 in. toward the outlet. The header flume extends around the room to the effluent port with stop-gate control to serve as a bypass for the aerator. This feature, together with stop-gate slots at the outlet for a closure plate and a mud valve in the floor leading to a waste connection, permits the cleaning of the aerator room without disrupting the flow through the plant. Screened, louvered window openings provide a fresh air intake while a 5,400 cu. ft. per min. roof ventilating exhaust fan for each aerator aids in the removal of liberated gases. The ventilating fan intake is led through concrete ducts to slots just above the water surface on the aerating room floor and opposite the windows to pick up the heavier-than-air carbon dioxide.

Rapid Mix

The rapid mix is carried out in two units each 16 ft. wide with 10 ft. of water depth. One is 40 ft. long and the other 42 ft. 9 in. Each basin is equipped with two "Dorrco" flocculator paddles, 7 ft. in diameter, running the full length of the tank and driven by a single-drive unit through a bevel gear to one shaft with intermediate chain-and-sprocket drive to the other shaft. Center baffle walls are provided to give around-the-end flow parallel to the flocculator paddle shafts. Each unit has a capacity of 6,618 cu. ft., giving a 7.1 min. retention period at 10 m.g.d.

The drives are equipped with 2-h.p. four-speed induction motors with moisture proof windings to give peripheral velocities to the paddles of from 0.55 ft. per sec. minimum to a maximum of 1.68 ft. per sec.

Flocculators

There are two flocculator basins 81 ft. 4 in. long by 42 ft. 9 in. wide by 15 ft. in water depth, each having a capacity of 50,569 cu. ft. with a 54.5-minute retention at 10 m.g.d. Each is equipped with five "Dorrco" flocculator paddles, 11 ft. in diameter, having a length approximately equal to the width of the basin. Each row of paddles is driven by a 1-h.p. four-speed motor with bevel-gear drive arranged to give peripheral velocities to the paddles of 0.55 ft. per sec. minimum to 1.61 ft. per sec. maximum.

Baffles are located between the rows of paddles and extend from the top of the tank to within 6 in. of the floor, with the exception of the last baffle, which is 1 ft. from the floor. Openings, 4 ft. by

6 ft. 9 in. are placed in alternate ends of all but the last baffle to cause the water to flow parallel to the paddle shafts. This reduces the tendency to short-circuit to a minimum. The openings beneath the baffles permit the return of a portion of the floc to the preceding flocculator to assist in flocculating the incoming water.

In the design of the basins, care was taken in the spacing of the paddles so the floc would be held in suspension continuously. The openings beneath the baffles permit the floc, which would ordinarily settle along the sides of a parallel type flocculator, to be swept back

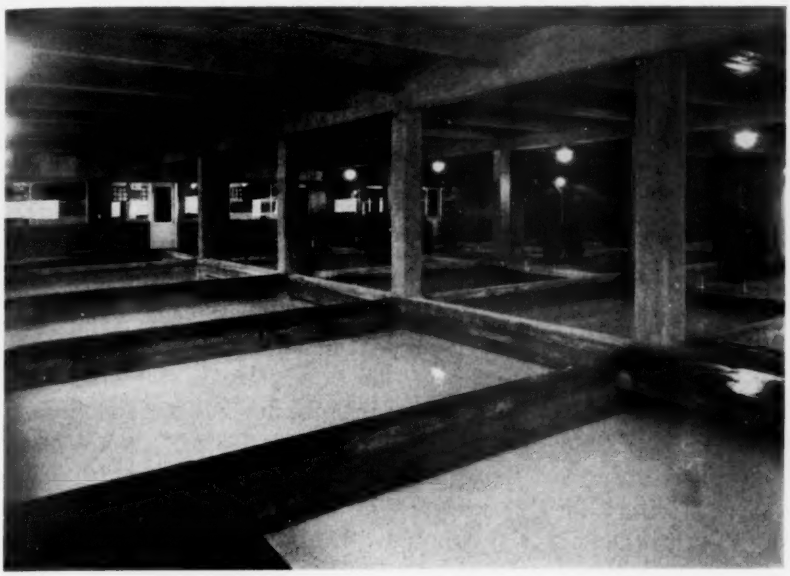

FIG. 8. Flocculation Room

toward the influent end of the basins. This prevents the accumulation of sludge along the baffle walls. As a result: (1) the efficiency of the units is increased by cutting out dead areas and permitting the entire tank volumes to be effective; and (2) the benefit of returned floc is obtained.

The water passes from the flocculation tanks to the clarification tanks through six openings, 2 ft. 6 in. wide by 5 ft. long, placed along the bottom and at the effluent ends of the tanks. Sloping baffles are placed directly over the openings to prevent the paddles from inducing a velocity into the settling basin.

First Settling and Carbonation Basins

The two first settling basins are located below the carbonating and mixing basins, and these are each 42 ft. wide by 136 ft. long by 19 ft. deep, holding 106,695 cu. ft. and having a retention time of 114.9 min. at 10 m.g.d. A 3-foot baffle wall is located near the entrance ports to prevent short circuiting along the top of the basin. Mechanical sludge collectors of the "Rex" conveyor type are provided in each basin for concentrating the sludge to a single draw-off sump and pipe in each basin. These were furnished by the Chain Belt Company of Milwaukee. Each basin is equipped with two longitudinal collectors equipped with steel scrapers, approximately 17 ft. long, and one cross collector, also equipped with steel scrapers, approximately 6 ft. long. This type of equipment, in addition to fitting into the plant layout advantageously, has certain desirable features of mechanical construction.

Steel channels (6 in. at 8.2 lb.) are used for scraper flights and are propelled by two chains. This construction provides very limited cantilevered construction of the scrapers. Chains are made of wear- and corrosion-resisting metal chosen for wear and durability rather than ultimate strength. The chains, being in tension and having large factors of safety, provide an extremely reliable form of sludge collector, designed with large factors of safety to withstand momentary overloads of considerable magnitude, and are adequate to meet every possible condition of operating service.

Shaft sizes were selected on a basis of conservative deflection values rather than for allowable stress. The sizes also provide large factors of safety in the parts. Bearings are babbitted and, because of the conservative shaft sizes, have low unit pressures on the bearing metal. For this service babbitt has proved entirely satisfactory since it is water-lubricated and since the shafts are revolving at an extremely low speed. Bearing construction provides for self-cleaning of any solids that might accumulate on top of the bearings and eventually become objectionable through decomposition.

Sprockets have chilled iron rims, providing accurate tooth form and hard wear-resisting surfaces in contact with the chains. Headshaft driving sprockets for cross collectors, and those adjacent to dividing walls on longitudinal collectors, are of dished design to allow duplication of shaft bearings and yet allow the passage of the driving chain without interference with the ends of the scraper flights at these points. All headshaft drive sprockets are of split

construction for easy renewal when replacement eventually becomes necessary.

Drive units are of motorized speed-reducer design, provided with 1725-r.p.m., $\frac{1}{4}$-h.p., totally enclosed motors with moisture proof windings. Units of this type have all gears completely enclosed and running in oil, which factors provide for a maximum efficiency and a minimum of operating attention. The longitudinal collectors have speed reducers with a ratio of 2,920:1, giving an output speed of 0.6 r.p.m. Longitudinal collector scrapers travel at 1 ft. per min. Similarly the cross collector speed reducers have an output speed of 1.2 r.p.m. Cross collector scrapers travel at 2 ft. per min.

These drive units are located on the operating floor and require unusually long chain drives for the head-shaft of the sludge collectors. In order to maintain the proper chain tension, the drive units are mounted on special bases, telescoping vertically and provided with easy means of adjustment, so that they can be used as chain tighteners when necessary.

Chain drives appeared to provide definite advantages over bevel gearing by allowing a more flexible choice of drive location, and also by eliminating the necessity for integral mounting of gears and pinions on the concrete structure, to preserve the alignment necessary for satisfactory operation. Furthermore, it was felt that since the drive chains could be observed at the drive unit sprockets, their condition, as to wear, could be determined at all times. With submerged gears, the gears, bearings, and thrust bearings could not readily be inspected, nor could their condition be determined without considerable difficulty and expense.

From an operating standpoint the sludge collector appeared to give the advantage of good design to meet all conditions that might be imposed upon it as well as having a low maintenance and operating cost and attractive appearance. From the standpoint of operating efficiency, it was felt that by moving the sludge directly to the draw-off pipe and by providing a considerable head of sludge there at all times, maximum concentration of sludge would be obtained and, therefore, there would be less water loss for sludge removal.

The carbonation basins are above the effluent ends of the first settling basins so that the influent enters the basin by 6 ports, 2 ft. 6 in. by 5 ft., in each basin distributed across the basin's width of 42 ft. 9 in. Each carbonation basin is 36 ft. 2 in. long with a water depth of 15 ft. and a capacity of 19,000 cu. ft. for a 20.5-minute re-

tention period at 10 m.g.d. Water leaves the basin through a series of slots 6 in. deep by 4 ft. long, four in number, leading into a collecting flume across the top of the basin. This throttled effluent serves to furnish uniform flow across the basin's width. Carbon dioxide gas is brought into the center of the basin, through a 4-inch cast iron main, connecting in the middle of a 4-inch header which extends each way across the width of the basin. Laterals lead 12 ft. in both directions from the header forming a grid of 1-inch wrought iron pipe on 24-inch centers and located 12 in. off the floor of the basin. The laterals are drilled with $\frac{3}{32}$-inch holes on 6-inch centers. The gate well on the effluent side of the basin and the covered access manhole on the influent side serve as gas domes for the exhaust gas. Four-inch vent pipes lead from the gas domes through the roof ending in a return bend with a screened opening.

Second Settling and First Recarbonation

Each second settling basin has a capacity of about 730,000 gal. in a basin 36 ft. wide by 109 ft. long with a water depth of 24 ft. 9 in., giving 103.3 min. retention time for 10 m.g.d. Uniform flow of water across the full width of the basin is maintained by influent and effluent slots, 5 in number, 5 ft. long by 4 in. high, distributed across the width of the basin and leading into a collecting flume—the influent at the bottom of the basin, the effluent at the top. No sludge collecting equipment is installed; sludge removal is to be accomplished by semi-annual draining and flushing. Two high pressure hose connections are provided in each basin for this purpose. Five feet below the effluent ports is a horizontal pipe grid suspended from the ceiling for first recarbonation. This grid consists of a central 3-inch cast iron header with $\frac{3}{4}$-inch wrought iron laterals 12 ft. long on each side spaced at 24-inch centers across the full width of the basin. Laterals are drilled with $\frac{3}{32}$-inch holes at 9-inch centers. Exhaust gases are vented by a 6-inch vent pipe leading from the gate well entrance of the filter influent flume to the roof. A baffle wall, extending from the ceiling of this flume, prevents the gas from carrying out into the filter room.

Filters

There are 6 filters, each having a flow capacity of 5 m.g.d. at 3 g.p.m. per sq. ft. Each filter box is 40 ft. 6 in. long, 32 ft. 8 in.

wide, 10 ft. deep and normally 8 ft. in water depth. The central 5 ft. is occupied by a wash water gullet. The sand surface area is 1,160 sq. ft., half on each side of the gullet. Four concrete wash

TABLE 6

Filter Valve and Gate Sizes

Filter Influent	20″ gate valve
Filter Effluent	16″ gate valve
Wash Drain	36″ x 36″ sluice gate
Wash Water	30″ gate valve
Surface Wash	8″ gate valve
Rewash	8″ gate valve
Rate of Flow Controller	16″

FIG. 9. Filter Gallery

water troughs 1 ft. 10 in. wide, spaced at centers of 8 ft. 2 in. lead into the gullet from each side. The lips of the wash water troughs provide a 27-inch freeboard above the surface of the sand. Four-inch cast iron collection laterals are spaced at 9-inch centers and

located with the centers 5 in. off the filter floor. The header for the laterals is under the central wash water gullet permitting wash water disposal at the back of the filter and filter effluent toward the front of the filter. Collection laterals have $\frac{1}{16}$-inch holes in the bottom at 6-inch centers.

An 8-inch header runs along each end of the filter basin for surface wash. The surface wash laterals parallel the wash water troughs and

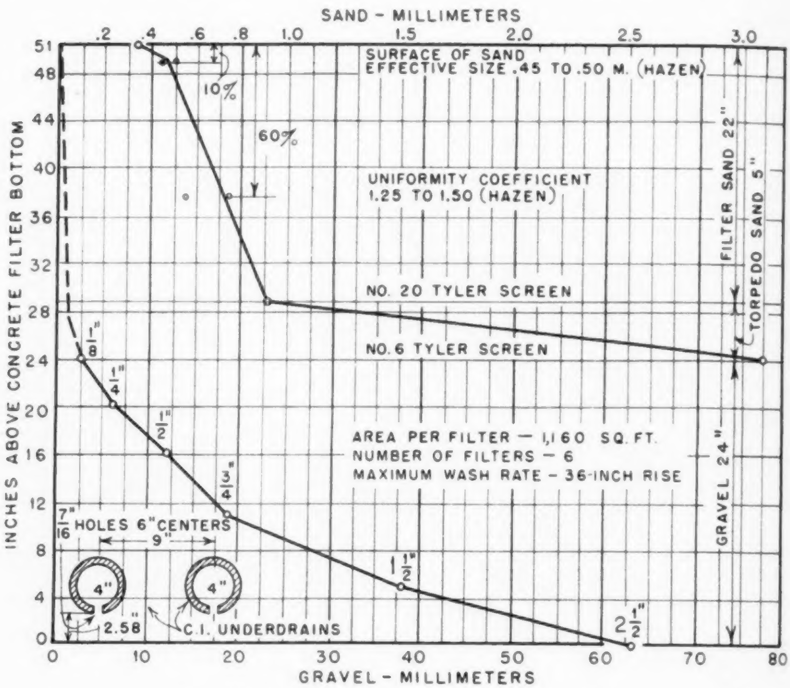

FIG. 10. Diagram of Sand and Gravel Sizes

consist of $1\frac{1}{2}$-inch galvanized iron pipe on 2 ft. 9 in. centers, drilled on both sides with $\frac{5}{32}$-inch holes at 8-inch centers. This surface wash grid is suspended from the bottom of the wash water troughs, 53 in. above the filter floor.

Total depth of sand and gravel is 51 in., made up of 24 in. of graded gravel from $2\frac{1}{2}$-inch to $\frac{1}{8}$-inch size, 5 in. of torpedo sand, and 22 in. of filter sand having an effective size of 0.45 to 0.50 mm. and a uniformity coefficient of 1.25 to 1.50 (see Fig. 10).

Final Recarbonation

Each of the two final recarbonation basins is 12 ft. 2 in. wide by 36 ft. 2 in. long by 15 ft. 9 in. deep; capacity is 5,446 cu. ft. and retention, 5.9 min. at 10 m.g.d. As the influent gate is in one corner of the basin, the flow is distributed across the width of the basin by a baffle plate in front of the gate and further through a distributing flume whose ports are covered with perforated plate. The effluent is taken from a collecting weir 2 ft. 9 in. from the ceiling across the width of the basin. The carbonation grid, located 1 ft. off the floor, consists of a central $2\frac{1}{2}$-inch cast iron header with 1-inch wrought iron laterals 13 ft. long extending both ways from the header and spaced at 18-inch centers. Laterals are drilled with $\frac{3}{32}$-inch holes at 6-inch centers. The basin is vented to the roof with a 4-inch vent leading from the covered access manhole which serves as a gas dome.

Chemical Handling and Storage

Ground conditions placed the main building some 150 ft. from the nearest railroad siding. The top of the storage bins is 70 ft. above the grade of the track. After due consideration it was decided that the proper way of unloading the necessary dry chemicals, lime, soda ash, and alum, from cars to bins would be pneumatically.

The Fuller Company of Catasauqua, Pa. was the successful bidder for supplying and installing the required equipment which has several distinct features. The unloading hose 5 in. in diameter, for a capacity of 10 tons per hr., is all metal with an overlapping hardened steel liner to present a smooth unbroken surface for the passing materials. For the complete retention and continuous delivery of dust coming from the dry treatment chemicals a four-compartment filter (81 in. in diameter) and receiver are used. A glazed porthole is provided in each compartment of the filter so that operator or visitors can readily see filter operations. The four compartments contain a multiplicity of tubular cloth bags that are successively cut out of service at definite fixed intervals for air reversal cleaning. The shaking of these out-of-service tubes, however, is variable, by means of a timer, from one to thirty minutes instantly changeable. For the convenience of the operator an electric clock is installed in the face of this remote timer. Unnecessary shaking of filter cloth, of course, lessens its life usefulness.

A 6-inch steel pipe line with "Dresser" couplings transports the chemicals from unloading hose to filter. A 50-h.p. motor at 1,160 r.p.m. operates, through a multiple V-belt drive, a "Connersville HD" type "RS" blower or exhauster with a capacity of 2,610 cu. ft. per min.

Distribution of these dry chemicals from the receiver to seven concrete storage bins is made by a 12-inch helicoid spiral conveyor equipped with motorized gates for remote control, open and closed indicating lights appearing on duplicate motor control panels.

The spiral conveyor is driven by a 5-h.p., 1,750-r.p.m. motor, through a "Falk" motor reducer, to provide a speed of about 60 r.p.m. The entire chemical handling system is electrically interlocked so that a storage bin gate must be opened before the spiral conveyor can be started or continue to operate. The conveyor, in turn, must be started and must continue in operation before the circuit can be closed for the exhauster.

The seven storage bins are equipped with "Fuller" high and low level diaphragm type bin signal indicators, the former connected with an alarm horn in the chemical unloading station at the railroad siding. Storage bins delivering directly to dry feed machines are equipped with "Fuller" manually operated 12 x 24-inch rotary gates.

The three lime storage bins are each 17 ft. 6 in. by 19 ft. 3 in. The three soda ash bins and the alum bin are each 17 ft. 6 in. by 6 ft. 6 in. All bins are 29 ft. deep. Total estimated storage capacity is about 700 tons or about a 30-day supply at 10 m.g.d.

Duplicate control and indicating light panels are installed, one on the bin floor and the other on the chemical feeder floor, and, in addition, a lighted panel board is installed at the car unloading position to indicate exactly to which bin delivery is being made.

Chemical Feeders

The complete and carefully drawn specifications covering the chemical feeding and slaking equipment were marked by the absence of "proprietary" specifications and by the rigid requirements enforced. In the specifications, stress was laid on the work to be accomplished by the machines and, in general, the details of construction were left to the manufacturer, so long as the design was adequate and proper material was used to meet the approval of the Board of Water and Electric Light Commissioners.

For instance, the requirements for the lime feeder and slaker were

detailed under the following headings in such a way that no proprietary or patented mechanisms were described, and so that any manufacturer who had equipment that would give the results specified could bid:

- Type of Equipment (Gravimetric)
- Kind of Lime
- Capacity of Feeder and Slaker
- Number Required
- Bin Connections
- Feeder Hoppers
- Feeder Mechanisms
- Feeder Housings
- Slaker Tanks
- Slaker Insulation
- Feeder & Slaker Drive
- Motors—Guards—Alarms—Painting
- Slaking Temperature
- Water Valves
- Slaker Agitators
- Short-Circuiting
- Grit & Stone Removal
- Grit Screen
- Dust & Vapor Removal
- Feed Adjustment
- Chemical Proportioner (Automatic)
- Feed Indicator and Totalizer

All feeders are belt type gravimetric feeders of the continuous type and are supplied through bin gates and short hoppers or chutes from large overhead storage bins. The following equipment was purchased from the Omega Machine Company of Kansas City, Mo.: 1 alum feeder, feeding range 2 to 150 lb. per hr.; 3 soda ash feeders, feeding range 75 to 1,125 lb. per hr.; and 3 lime feeders and slakers, maximum capacity 3,000 lb. per hr.

The equipment has a total nominal maximum feeding range of 12,500 lb. of chemical per hr. The maximum capacity of all feeders, however, will never be required at one time.

All feeders are alike, with interchangeable parts, and any feeder can be made to duplicate the feeding range of any other feeder. Certain of them are constructed right and left hand in order to obtain a more symmetrical installation.

Special Installation Features

There are several special features of the installation as a whole, as well as of the chemical feeders themselves, that are noteworthy:

Headroom: Despite the extremely high range required of some of the equipment, the feeders, complete with their bin gates and con-

necting chutes, were installed under concrete storage bins, with their bottoms only 7 ft. 10 in. above the floor on which the equipment rests. The low headroom required results in a considerable saving in building height where large chemical storage bins are constructed above the feeding equipment.

Discharge Troughs: To overcome difficulties due to corrosion or stoppages of chemical discharge pipes, a system of troughs was worked out whereby the solution or suspension discharged from the mixing chambers is carried to the point of application by means of open conduits. These conduits are covered by hinged cover plates, but can be opened for inspection for their entire length at any time and can easily be flushed out.

It is believed that the difficulties due to the "liming up" that is usually experienced with lime discharge lines have been completely eliminated.

Appearance: An effort has been made to arrange equipment symmetrically and to provide neat-appearing equipment. The doors to the feeding compartments of the chemical feeders and to the operating compartments are plate glass, and the interiors of these two compartments have concealed illumination. Thus, the operator can tell at a glance without opening the doors to the feeder whether or not it is operating properly.

Accuracy: The specifications require that the feeders deliver with an accuracy or uniformity of one per cent plus or minus. A special test beam, which can be used for checking the rate of delivery, is built into the scale. The totalizer records directly in pounds the total amount of material that has passed through the feeder. It is not necessary to apply any factors to obtain the readings in pounds, regardless of the rate of feed.

Alarms: All feeders are provided with an audible alarm which will operate if anything happens to interfere with the rate of feed.

Automatic Proportional Control: The rate of feed of the chemical feeders is automatically controlled by either one of the two Venturi meters bringing the raw water into the plant. The control is of the variable time interval type, i.e. with maximum flow, the feeders operate continuously; at 50 per cent flow they operate for 50 per cent of the time; at 25 per cent flow they operate for 25 per cent of the time; etc.

The operating cycle for the feeders is 15 seconds, so that at $\frac{1}{4}$ flow the feeders will be on for 15 seconds and off for 45 seconds.

To level out any irregularities in chemical dosage which might possibly be felt, dissolving chambers of ample capacity are provided so that the material discharged from them is of practically constant strength. The lime slakers, for instance, have a 30-minute retention period, so that the intermittent feed is not discernible at the slaker outlets.

Slakers: Each of the lime slakers has three compartments so that short-circuiting of material from inlet to outlet is prevented. Two 2-h.p. motors, each driving two 6-inch propellers, provide agitation for mixing. Slakers are completely insulated on all sides and top and bottom with 1 in. of 85 per cent magnesia. An extra heat exchanger compartment is provided and in it part of the heat from the outgoing lime is recovered by the make-up water used in slaking. Slaking temperatures are recorded on 24-hour charts. Temperatures are maintained between 150° and 180°F. by thermostat arrangement on the water supply. The water supply is taken at 50°F.; 20° increase is secured from the heat exchanger; 10° increase, from the dust and vapor removal equipment; and the balance, as much as 100°, from slaking.

Dust and Vapor Removal: Dust and vapor removal of ample capacity is provided so that the equipment may be operated up to 200°F. without the escape of vapor or dust.

Sludge Return: Two 2 x $1\frac{1}{2}$-inch centrifugal pumps rated at 75 g.p.m. with a 33-foot head, direct-connected with $1\frac{1}{2}$-h.p. motors at 1,150 r.p.m. are connected with the sludge lines and discharge, as desired, through valved piping at each or any lime slaker discharge. In addition to the improved floc formation by the addition of sludge to the feeding chemical, the added volume serves to keep the lime feed trough flushed out.

Hydraulic Control

Control equipment is actuated to a large extent by air pressures rather than by hydraulic or electrical means. This provides advantages in: (1) continued cleanliness of operating mechanism; (2) smaller tubing and fittings for control; (3) increased speed and improved sensitiveness of response to changes; and (4) more flexibility in location of equipment. An air flow indicator with pressure gage and regulating valve is located at each air distributing point to provide only that amount required for each piece of control equipment. A motor-driven compressor, with a storage tank of 8.5 cu. ft.

capacity at 100 lb. pressure, delivers about $1\frac{1}{2}$ cu. ft. of air per min. at 30 lb. pressure to the supply mains for this control. A smaller compressor set to cut in at 25 lb. pressure serves as a stand-by unit.

At the throat of each of the two 24-inch Venturi tubes for the raw water entering the plant is a 14-inch hydraulic valve for automatic emergency control of high water levels and for regulating the amount of water conditioned by each side of the plant. The automatic control is governed by an air pipe having its tip submerged when the water level in the flocculating basin rises to within $7\frac{1}{2}$ in. of the operating floor. Submergence of the air tip produces a back pressure which is piped to sealed oil-float units at the hydraulic valve which is operated through the floating lever and pilot valve groups. Full closure of the valve would be effected if the water should rise to within 3 in. of the operating floor. Basin level indicators located on the mechanical operating panel are operated by this same back pressure, or the pressure can be artificially induced at the panel for the manual control of these valves. This equipment is normally inoperative as the well field is closed to meet the reservoir demand rather than throttling the pumpage on the entire field.

Raw water flows are indicated on the mechanical panel through air differential pressures developed at the Venturi tubes. The indicating gage for this flow operates a cyclometer device which, in turn, electrically actuates the dry feeders for automatic proportional control.

Automatic controls regulate the filtered water delivery rate to match the raw water supply rate through the water level of the flocculation basin. Provision is also made for manual control of the total filter rate from the master control table and for individual operation of each filter unit independent of either the automatic-master control or the manual-master control. This control is transmitted to the filter rate of flow controllers by the application of air pressure to liquid sealed bells, which action induces a change in moment on the controller lever beam. The rate-of-flow controllers are direct-acting, guillotine-valve type installed in a vertical position with the controller beam balanced by a dead weight to overcome the inertia and weight of the actuating equipment as against the air bell and the pull of the water Venturi head acting upon a long-stroke constant area diaphragm pot.

For automatic control of the filters, the head of the water in the flocculation basin from a level of $7\frac{1}{2}$ in. below the operating floor to 24 in. below is utilized. This head proportions the flow of water

through an orifice at the lower level, the flow varying as the square root of the basin level above the orifice. To locate the equipment at a central point, at the master control table, the regulating head of the two flocculation basins is transferred under the table by a 2-inch pipe leading out of the stilling wells. The proportioned water flows through a catch basin to a head tank which has a solenoid-controlled valve outlet of calibrated opening for each filter, the outlet valve being electrically opened for each filter in service as controlled by a mercoid switch on the filter effluent valve. By this means the level of water in the head tank will be a measure of the rate at which those filters in service will need to operate to match the incoming rate; a gage glass on the head tank is calibrated for this rate. The head tank converts the total range of $16\frac{1}{2}$ in. of the flocculating basins to a range of $52\frac{1}{2}$ in. for minimum to maximum flows. Through an air tip in the head tank this range in head is converted to an air back-pressure which is transmitted to the controller air bells and also to an indicating rate scale on the master control table.

For manual-master control, valves on the master operating table permit the closing of the head supply from the flocculation basin and the opening of a water supply to establish any desired head in the head tank and a proportionate filter rate. Also, by having a valve to bleed off water from the head tank, wide flexibility of the equipment is permitted in adding or subtracting water from the head tank from the flocculation basin either under manual or automatic control. Increased temporary response to filter rate changes may be accomplished by an air increase and decrease valve on the master operating table in the air tip line from the head tank to the controller air bells.

For the individual control of filters, each filter has an air switch valve for automatic control (with the air pressure regulated by the master control, either manual or automatic) or for regulating the air pressure to the controller bell directly.

Rate and loss gages for the filters are operated by the actuating mechanism and air operating displacer tanks located under the operating tables. Differential air pressures equal to loss of head and Venturi head are produced by baffled air-venting chambers in the pipe gallery which connect with: (1) the raw water on the filter; (2) the main section of Venturi of the controller; and (3) the throat section of the Venturi. Checks operated when the wash drain valve is opened prevent wash water pressures from influencing the gages

during filter backwashing. The gages are of the 18-day strip-chart type with a feed of $\frac{1}{4}$ in. per hour. The rates of flow of all filter gages are totalized automatically, then indicated on a 24-inch dial gage in the filter gallery and recorded at the mechanical panel. Summation is accomplished by displacer units with "carbitol acetate," a non-evaporating liquid, being added or subtracted in accordance with the rate of each filter.

Wash water rate is not measured, the control for filter backwashing being measured by sand expansion. Each filter is equipped with a "Herring" sand expansion float mounted over the sand near the front of the filter wall. The device will measure up to 75 per cent expansion, indicating the amount on a dial mounted on the operating table.

The clear water reservoir and wash water tank contents are indicated on 24-inch illuminated dials (in the filter gallery) operated by long-stroke constant area diaphragm-spring units responsive to the back pressure from the air tips. On the mechanical panel, in addition to the items previously mentioned, are: (1) an air-operated wash water tank volume recorder; (2) an air-operated indicating wash water reclaim basin level gage; (3) five alarm lights operated by electric float switches connected to each of the flocculating basins, wash water tank, clear water reservoir, and reclaim basin; and (4) an alarm horn connected to the signal circuits of 3.

All hydraulic control equipment was furnished by the Simplex Valve and Meter Co. of Philadelphia.

Pipe Gallery

The pipe gallery is 17 ft. wide. With vertical rate-of-flow controllers and with the wash water piping and valves suspended from the ceiling, the gallery has a clear width between the outside limits of the rate-of-flow controllers of 6 ft. and an overhead clearance of 9 ft. to the wash water pipe. The gallery has a grade entrance with glazed doors for natural lighting and, in addition, is artificially lighted. The gallery floor is well pitched to gutters along each side. Leakage is nil. All these factors contribute to provide a pipe gallery that is light and clean.

Carbon Dioxide Generating Equipment

Equipment to produce carbon dioxide gas for use in the water conditioning process is furnished in duplicate, each unit being capable of producing 6,000 lb. of carbon dioxide per 24 hr. The equipment

consists of coke-burning stokers, boilers, compressors, scrubbers, driers, CO_2 per cent recorder, mechanical flow meters, and automatic control equipment for constant percentage and pressure of gas. In addition, of course, is the necessary piping and valves for conveying the gas from the producers located at the Cedar Street Pumping Station to the carbonating basins at the plant.

The stokers, manufactured by Holcomb & Hoke, are automatic electric motor-driven ones of the screw type underfeed, each being capable of burning a minimum of 10 lb. to a maximum of 90 lb. of coke per hour.

The gear case which contains the fuel feed drive, consisting of two heavy bronze worm wheels and two specially hardened and ground nickel steel worms all mounted on annular ball bearings, runs in a bath of lubricant at all times.

Each stoker is equipped with a feed hopper built of heavy copper-bearing steel plates, heavily welded. Coke is fed by gravity to the feed worm.

The feed worm is a one-piece casting of special alloy steel extending from the gear box into the retort.

Fuel feed drive is by a small electric motor mounted high on top of the transmission case. This type of mounting protects the motor from water or other injury.

The retort is also a one-piece casting. It is constructed to give an even amount of fire on both sides of the boiler.

The tuyeres are designed to give maximum, unrestricted air passage to the fuel bed. Being unusually high, they give a sloping fire that is clean at all times, as the slope is so proportioned as to cause all incombustible waste to flow readily to the sides of the fire box, where it cools in the form of clinkers and is easily removed.

The boilers are "Murray Iron Works" 20-h.p. units constructed for a 200-lb. working pressure to comply with the A.S.M.E. Code. These boilers are of the horizontal-return tubular type. The boiler is carried on structural steel columns entirely independent of the brickwork.

These boilers are furnished with a steel casing made of heavy steel plate reinforced in the corners and on the bottom with heavy angle irons. The steel casing is part of the boiler and is lined with fire brick backed with sufficient insulation. This type of setting is very economical, having long life and being easy and inexpensive to replace. The fire box or producer is of large volume as is necessary

for complete combustion and for a high percentage of CO_2, the principal aim of the coke-burning equipment. The casing and boiler are well adapted for stoker equipment and firing.

Each boiler was furnished with the following auxiliary equipment:

One 250-lb. Swartout "SM" Feedwater Regulator with stainless steel valve trim.

One Yarway tandem blow-off valve consisting of one iron-body alloy-fitted flanged Yarway seatless straight-through valve and one Yarway double tightening flanged valve.

One Lunkenheimer stop-and-check valve designed for 250 lb. steam working pressure.

One set of Lunkenheimer glove-feed-inlet and feed-inlet swing-check valves designed for 250 lb. steam working pressure.

Two Lunkenheimer iron-body bronze-fitted safety valves designed for 250 lb. steam working pressure.

One Wright-Austin water column with low water alarm.

The boiler was covered with 85 per cent magnesia insulation and the steel casing was lined with 2-inch "Webers 48 Insulation Blocks."

The compressors are of the "Nash Engineering" rotary water-seal type, one driven by a 30-h.p. steam engine, the other by a 30-h.p. electric motor. Each compressor has a rated capacity of 330 cu. ft. per min. at a speed of 865 r.p.m. and with a discharge pressure of 12 lb. per sq. in. Speed adjustment provides a delivery of one-half maximum to full maximum capacity.

The motor-driven compressor is equipped with a hydraulic coupling which makes it possible to obtain an infinite number of different speeds between minimum and maximum rates.

Each of the scrubbers is constructed of reinforced concrete 72 in. in diameter by 10 ft. high. They are made up in two sections, both top and bottom halves being provided with removable handhole covers making possible quick access to the interior of the scrubbers for inspection and repairs.

A spray pipe system, from which fine sprays of water are discharged onto the limestone below, is also provided in the top of the scrubbers. As the gases are pulled upward, they are cooled and washed by the water passing through the limestone.

The acid products are carried down to the bottom of the scrubber

with the water and there they are trapped out of the system while the cooled gases are drawn off the top of the scrubber.

Each scrubber is equipped with the necessary thermometer, sample cocks, and pressure gages.

"Hays" regulatory equipment provides for a constant percentage and pressure of the generated gases. The pressure at the receiver actuates a master control which controls the compressor speed, the stoker feed and the amount of induced draft for the stokers. The compressor speeds on the motor driven unit are controlled by an oil operating cylinder with a pilot valve arrangement acting on the arm of the hydraulic coupling. The steam engine speeds are regulated by throttling the steam line.

The pressure gages and the CO_2 per cent-and-flow recorder are installed at the pumping station for control of carbon dioxide generation. The mechanical panel at the conditioning plant is provided with a "Brown" flow meter for recording total flow, a "Brown" electric CO_2 per cent recorder, a "Republic" flow meter for indicating the pressure, temperature and flow in pounds per day for each of the six carbonating basins, and control dials for the "Powers" air-operated regulating valves in the CO_2 mains.

Wash Water

The wash water tank is of all-welded-steel construction, 23 ft. 6 in. in diameter and 33 ft. high, and has a capacity of 104,000 gallons. The minimum head provided is 25 ft. 6 in. above the lip of the wash water troughs. The tank is constructed with a $\frac{3}{8}$-inch bed plate resting on a sand and asphalt cushion, with the sides consisting of 80-inch plate rings, the bottom of $\frac{3}{8}$-inch plate, the next four rings of $\frac{5}{16}$-inch plate, and the top overflow skirt of $\frac{1}{4}$-inch plate. The downcomer is 30-inch welded steel pipe of $\frac{5}{16}$-inch plate. The wash water header in the pipe gallery is supported by hangers from above leaving the pipe gallery clear of supports. Wash water is provided by an 8 x 8-inch centrifugal pump rated at 1,800 g.p.m. at a 55-foot head, direct connected, with a 30-h.p. standard open induction motor of 1,750 r.p.m. Wash water supply is taken from the filter effluent flume. A cross connection permits the filling of the wash water tank direct from the high pressure distribution system with a similar cross connection for the filter surface wash. Normally, surface wash water is taken from the wash water tank. Automatic operation of

the wash water pump is provided by high and low electrodes in the tank.

Filter wash water is reclaimed through a basin of sufficient capacity for storage for two filter backwashes. The basin is located directly below the pump room floor. In it electrode controls operate two pumps for staggered operation—one pump for low levels and both pumps for the higher levels. The pumps are each 4 x 3-inch centrifugal pumps rated at 350 g.p.m. at a 40-foot head with 5-h.p. standard open induction motors of 1,740 r.p.m. They have a common discharge which is split for returning the water to the head of either or both flocculating basins. Valved piping permits the waste of wash water through free discharges to either the sanitary or storm sewers. Wash-water-reclaim basin overflow, of sufficient size to take the full backwash rate, connects with the storm sewer through a free discharge and trap arrangement.

Through the system of gravity drains for the plant there is a valved pipe connection to the wash-water-reclaim pump suction line. This permits reclaiming the water in any particular basin by pumpage to the opposite unit of the plant. Also, this same connection permits complete dewatering of the plant as its bottom is below the grade of the gravity-drain storm sewer connection.

Filter Operating Tables

Filter operating tables enclose concealed heating units as well as filter control and metering equipment. The table front is constructed of aluminum, using chiefly standard aluminum-rolled and extruded shapes. The finish is alumilited aluminum with contrasting bands of black alumilite. The table top is of ebonized asbestos with the mechanical parts on the table top of aluminum with natural alumilited finish, made up in an octagonal pattern. Operating handles are modernistic, of the automobile type. Indicating dials are made of glass in two color combinations. The faces of the rate-and-loss meter and the sand-expansion indicator are inclined to the surface of the operating table for ease in reading and for prevention of errors due to parallax. Electric switches on the top provide for indirect lighting of the indicators and meters and also for two 1,500-watt lights to floodlight the filter. Filter-control-table operating equipment was furnished by F. B. Leopold Co., Pittsburgh.

Sampling and Testing

To aid in the operation and control of the plant and because of the inaccessibility of some of the basins, $\frac{3}{8}$-inch copper tubing has been run to 19 different points in the plant leading to a common sampling table. This table is provided with 3 illuminated sampling glasses for drawing and wasting or for holding a quart sample by means of an aspirator connection. An air supply line permits the cleaning of any sample line after the sample has been drawn. The sampling room has no window openings so that the lighting, with fluorescent lights, is uniform to permit uniform results with colorimetric indicators on the titration tests.

In addition to this dark room for sampling and titrations, a well lighted and ventilated laboratory, 24 ft. by 25 ft., provides ample room for complete analysis and research work. An office desk, a table for analytical balance, a desiccator, a drying oven, an 8-foot chemical storage cabinet and counter, a glassware cabinet, and 50 ft. of work-bench space are provided. Work benches have convenient outlets for electricity, compressed air, natural gas, carbon dioxide, aspirator, and hot and cold water. Some of the equipment that adds to the completeness of the laboratory are a centrifuge, a pH meter, a muffle furnace, two 9 x 13-inch hot plates, a one-gallon-per-hour still, a 6-place "P. & B." laboratory mixer, a single stirring motor with flexible shaft, an electric laboratory shaker, a microscope, a steam bath and fluorescent lights.

Sludge Disposal

The digester capacity of the Lansing Sewage Disposal Plant was insufficient to take the load of the water conditioning plant sludge and the Michigan Stream Control Commission objected to its disposition in the Grand River, so it was necessary to resort to disposition by pondage. Low value lowlands in the southeastern part of the city adjacent to the Riverside air-lift-well field have been diked to provide three ponds, about $5\frac{3}{4}$ acres in each pond. The sludge line outlet at the first pond is 7,800 ft. from the plant, with the third pond 8,900 ft. from the plant. The sludge line is made up of 6-inch cement-asbestos pipe with couplings and cast iron pipe for the fittings. The route of the line, for the most part, parallels the Pere Marquette Railroad and it is believed that the type of pipe and joints selected provide the best insurance against leakage caused by shock

and settlement through the flexibility of the joints. Cement-asbestos pipe has been rated as offering less frictional resistance to flow, a decided advantage in a line of this length and in one carrying a material that has a tendency to settle out. Manholes have been placed at all bends in the line connected with the couplings, a feature which provides easy access for maintenance inspection and cleaning.

As noted, the two high points in the line are equipped with air vent valves with vacuum seals, and the pondage outlets are submerged so that the line will flow full at all times and get the maximum benefit from syphon action. The sludge is pumped by two 2 x 2-inch centrifugal pumps having a rated capacity of 150 g.p.m. at a 35-foot

TABLE 7

Control Elevations on Sludge Line

POINT	ELEVATION
Hydraulic gradient over sludge sump (min.)	856.0
Sludge intake, 4″, to each sump	817.0
Sludge pumps located in pump room	835.5
Plant sludge outlet	832.0
Sludge line station 41—air vent valve	860.2
Sludge line station 65	850.2
Sludge line station 70—air vent valve	854.8
Sludge line station 78—first pond outlet	834.4
Sludge line station 89—third pond outlet	827.1

head and powered by 3-h.p. motors of 1,740 r.p.m. These pumps are so valved that each may pump from its respective sump or so both may pump from one. They have by-pass connections for discharge to a sanitary sewer, to a storm sewer, and to the sludge ponds for syphon flow. A high pressure connection permits dilution of the sludge or cleaning of the sludge line. A Venturi meter installed in the line at the plant furnishes a measure of the pipe flow which, with sampling cocks, provides a means for determination of the amount of sludge and carrying-water removed from the plant.

Construction

The water conditioning plant was built as a project of the Works Progress Administration with the City of Lansing, through its Board of Water and Electric Light Commissioners, as the sponsor. Work was started in April, 1938, and the plant was placed in operation in December, 1939.

To maintain the utilities at the site of the project and to do certain types of skilled work, board employees and W.P.A. labor worked together. Construction procedures often required continuous workings or the use of labor beyond the maximum number of hours permitted under W.P.A. The majority of the workmen furnished under W.P.A. were not familiar with the application of their trade in connection with building construction. There was also a decided shortage of skilled workmen under the W.P.A. classification as applied to the project, many of the men being skilled in other trades or not skilled at all so that they had to be classified as common labor. In spite of all these difficulties, through the close cooperation of the State and District officials of the W.P.A., harmony and rapid and efficient prosecution of the work prevailed at all times. While the fundamental purpose of W.P.A. projects is to provide employment, and despite the consequent substitution of hand methods in the performance of many phases of the work, at no time was there any "make-work" attitude to impair the business efficiency of the job. Where machine methods could be used to advantage they were so used. It is believed that this procedure reflected favorably on the conduct of the men—the readily apparent results of their efforts making them more willing to work.

The structure required 11,080 cu.yd. of concrete. This was mixed at a central mixing plant, lifted by a tower bucket hoist, chuted to a hopper near the pour location, and placed in the forms with concrete buggies. Chutes were eliminated on the pours at the high elevations, but for the majority of the work this method offered the best solution for work carried on in the multiple stage construction. In the procedure the concrete was placed within the barriers formed by the walls and the reinforcing steel of the basins. Segregation was minimized by the collection hopper and by placement with concrete buggies. The mixer was a half-yard motor-driven unit which had ample capacity for the average pour of about 60 cu.yd. and the maximum pour of more than twice this amount.

All aggregates were measured by weight, with the mix determination controlled by the water-cement ratio; no admixtures were used. A dense and watertight concrete was secured by careful proportioning to produce a "workable" mix and by the use of a liberal amount of labor to place and spade the concrete in the forms. An electrical concrete vibrator was used in placing concrete but its best application appeared to be in use on the slabs rather than in wall construction.

Field design mix was based on a minimum of 3,000 lb. of concrete for 28-day strength. Fine and coarse aggregates, as produced by the local pits, were screened to meet Highway Department Specifications and were deficient in "fines," in each case, for watertight basin and building construction. This condition was remedied by separate stock piling and weighing, for each batch, of a fine sand and pea gravel to add to the fine and coarse aggregates respectively.

In addition to compression cylinders for a check on the strength of the concrete, plain concrete beams were cast, 6 in. x 8 in. x 36 in., for the determination of the modulus of rupture through beam breaks. Control by beam breaks provides a convenient method of field

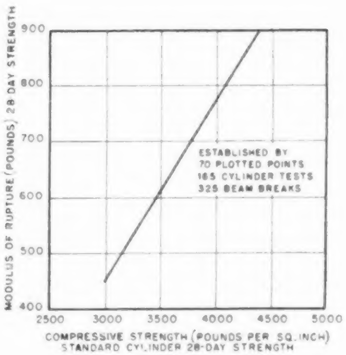

FIG. 11

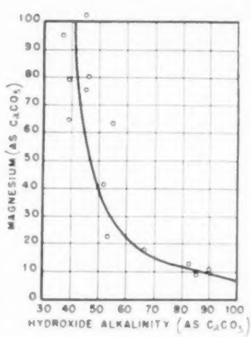

FIG. 12

FIG. 11. Ratio between Modulus of Rupture and Compressive Strength of Concrete

FIG. 12. Hydroxide Alkalinity and Residual Magnesium

determination. The beams are placed in a clamped holding arrangement and loaded with an hydraulic jack recording the load necessary to break the beam. A 12-inch moment arm is used so that each beam cast is good for two breaks. Having used both the compression cylinders and beam breaks on this project, a relationship has been established between the two (see Fig. 11) so that future work may be controlled by beam breaks alone. This relationship checks quite closely at the lower limit with the curve in the *Concrete Pavement Manual*, published by the Portland Cement Association, but the Lansing results are much more conservative in the higher brackets. Concrete strengths, as determined from the compression cylinders, ranged from 3,000 to 4,500 lb. per sq. in. at 28 days.

Of the 345,000 sq. ft. of form work required, nearly half was done with "Economy Steel Forms"; a steel form furnished in several sizes for make up and re-use as might be required. Wooden forms were tied with steel pencil rods which were cut off in back of the concrete surface. Plaster waste molds, provided in sections, were used for the sculptured figure.

Concrete slabs were cured by ponding, walls were cured by continual wetting of the forms. Construction was continued through the winter of 1938–39 with the concrete aggregates and water heated so that the concrete temperature as placed in the forms was about 70°F. Concrete poured in the winter was housed with tarpaulins and heated both with steam and salamanders. Steam jets permitted the direct escape of steam for proper humidity control.

Reinforcing steel bar chairs were made of small tapered concrete blocks. Thus, no metal which might cause rust spots was exposed. The concrete blocks blend in with the pour so that they are not discernible after the finishing operation. The concrete was finished by carborundum bricking of the surface without plastering. Power grinders were furnished for this purpose.

The exterior of the structure below ground level was given two applications of "R.I.W." marine cement and the inside of all water-containing basin walls or floors wherein the opposite face of the wall or floor is exposed, either inside or outside, were painted with two coats of "Waterlox" transparent waterproofing. All construction joints exposed to water were cast with "No. 22" galvanized "Armco" iron-folded waterstop. In addition, all construction joints were given two coats of "Waterlox" waterproofing extending at least one foot on each side of the joint.

To close in the structure as much as possible before extreme winter weather, the bottoms of the chemical storage bins were omitted, to be poured after the structure was completed. While unusual, the procedure worked successfully as the concrete was brought in the window openings and spouted to the bin bottoms through the opening left for the chemical filling. Filling gates on the forms extended above the pour, the excess concrete being chipped off after it had set. The filter wash water troughs are of concrete and were cast in place. The lips of the troughs were slightly cambered during casting to offset deflection. After the troughs had set, the filters were filled with water and the lips of the troughs were bricked with carborundum brick until the water broke uniformly throughout the filter.

Local gravel pits provide a gravel and sand which was found by experiment to be comparable with the gravels and sands produced especially for filter purposes. The chief difference lay in the acid loss of the sand, such loss running as high as 24 per cent. The experiments indicated, however, that this condition would not be detrimental in the plant since the water entering the filter is stable or has a plus stability index. Then, too, even with aggressive water the loss would be very slight. Accordingly, local gravel and sand were used. While the gravel was screened at the pit as to size requirement, the percentage of undersize material was too great to allow use without additional screening on the site. The gravel was shoveled on an inclined screen with openings of the next size larger than that desired for the smallest size; viz. a $\frac{5}{8}$-inch screen for $\frac{3}{4}$-inch to $\frac{1}{2}$-inch material. The inclination of the screen reduced the effective size of the openings so that the resultant screened material conformed closely to the size desired. The fines or screenings from each size were placed in the filter to make the first portion of the next and finer gravel-size layer as these screenings bordered on the larger size gravel for this layer; in other words, the screenings of the $1\frac{1}{2}$- to $\frac{3}{4}$-inch stone were close to $\frac{3}{4}$ in. in size and could be used as the bed of the $\frac{3}{4}$- to $\frac{1}{2}$-inch layer. Through this process the filters had a much better gradation from coarse to fine than would normally have been the case. All of the larger stone down to $\frac{3}{4}$ in. in size was hand picked for the removal of sandstone, chert, and elongated pieces. For the production of filter sand, two screenings were required—a coarse screen for the control of uniformity coefficient and a fine screen for the control of effective size. For coarse screening, 14-inch mesh window screen proved to be the correct size. Laboratory screening used a 16-inch mesh screen for the separation, but the field screen was again an inclined screen set at about 45 degrees to reduce the effective size of the openings. Sand taken from the top of the coarse screen furnished the material for the torpedo sand bed in the filter. The screenings were passed over a 30-inch mesh screen with openings equal to the desired effective size (.475 mm.) to remove the fines. For this fine screening a mechanical riddle was made, consisting of a 12-foot length of screen set on a slope of about 1 to 12. The high end was agitated by eccentrics with a 1-inch throw mounted on a shaft turning at about 250 r.p.m. About 10 per cent of the fines carried over so that the resultant filter sand had the effective size of the screen openings. To permit the screening, the sand was dried

on "dutch ovens," using scrap form-lumber for the fires. The entire procedure was set up for continuous operation employing four shifts of labor working six hours each. The local sand yielded about 18 per cent torpedo sand and 38 per cent filter sand. Some of the waste fines were stock piled for the use of the service department in making backfills under pavement excavations, and the bulk of the remainder was mixed with coarser material to be used by the city for the sanding of icy streets.

The sand was dumped into the filters from wheelbarrows at the operating floor elevation. The compaction of the sand dropped from this height, and the subsequent working to spread it in place, was such that after repeated backwashing, the bed had expanded from 6 to 9 per cent.

TABLE 8

Project Costs

ITEM	FEDERAL CONTRIBUTION	BOARD OF WATER AND ELECTRIC LIGHT COM'R.
Labor	$425,259	$170,601
Material	79,535	359,806
Total	$504,794	$530,407
Grand Total	$1,034,141	
Regular employees	$78,489	
Extra labor	92,112	
	$170,601	

In the construction of the plant several items calling for specialized trades were performed under separate contract arrangements. These contracts all provided that common labor would be furnished by the W.P.A. Several such contracts were for: roofing, plastering, terrazzo floor, and for installation of chemical handling, chemical feed, flocculating, and sludge collection equipment.

The total cost of the project is allocated as shown in Table 8.

Operation

The plant's operating period has extended only a few months; consequently no conclusions can be drawn at this time. An apparent trend, however, may be pointed out.

Prior to opening, the plant was operated at a 2-m.g.d. rate for about 2 weeks. During this time a high chlorine residual was carried in the water to disinfect the basins, and the effluent was run to waste after first being employed in filter backwashing. Through this process the running order of equipment was established, the operators secured familiarity with their duties, control equipment was adjusted and the entire plant was given a short seasoning period. No gradual transition from the hard to soft water supply was made other than through the dilution of the softened supply in the reservoir and distribution mains. The entire water production was softened to 85 p.p.m. hardness when the plant was "cut-in" and is being held at that softness.

Before putting the plant in operation, a check on leakage in this concrete structure (exclusive of sluice gate and valve leakage) was made (see Table 9).

TABLE 9

Plant Leakage

Holding capacity of plant (exclusive of final recarbonation), *mil. gal.*	4.57
Total water-concrete contact area, *sq. ft.*	60,074
Water depth in structure at wall, *ft.*	$34\frac{1}{3}$
Leakage, *in. per day*	0.26
Leakage, *g.p.d.*	2,184
Leakage per million gallons capacity, *g.p.d.*	478
Leakage per 1,000 sq. ft. concrete area, *g.p.d.*	36.4

The water tightness compares very favorably with other concrete water-containing basins, especially in consideration of the large hydraulic head in this structure.

Operations of the plant have not reached their optimum condition but there is progress as shown by Tables 10 and 11.

The figures under the heading "Expected Results" are based partly on theory and experiment, and partly on plant results. As the heading indicates, the figures are subject to change; for example, if too much incrustation of the filter sand is experienced, more CO_2 may be applied in the first recarbonation, thus lowering the pH and phenolphthalein alkalinity slightly to supply a more stable water to the filters. Similarly it may be found that better flocculation and magnesium removal will be obtained if the hydroxide alkalinity in the first settling basins is increased to 70 or 80 p.p.m. The whole

picture from this point on would be changed as a result. From the reaction record for February and March it can be seen that the average hydroxide alkalinity is being increased, but it is still short of the goal of 60 p.p.m.

Experiments on the proper hydroxide alkalinity to carry in the first settling basins for the best magnesium removal and flocculation are now being made. Short runs with increased lime feed to give 60 p.p.m. hydroxide alkalinity have given much lower turbidities and residual magnesium in the water from the first settling basins. Small scale experiments, made before the plant was placed in operation, indicate the possibilities for magnesium removal by excess

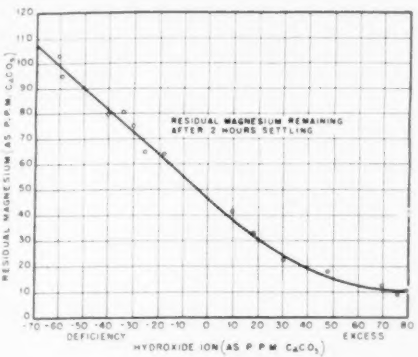

FIG. 13

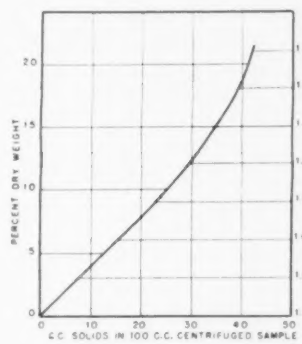

FIG. 14

FIG. 13. Effect of Hydroxide Ion on the Amount of Residual Magnesium
FIG. 14. Per Cent Dry Weight and Specific Gravity of Sludge; as determined by centrifuged sample

lime treatment. Figure 12 shows the trend as evidenced by plotting residual magnesium against hydroxide alkalinity.

The total hydroxide alkalinity may be considered to include both the excess calcium hydroxide and the residual magnesium hydroxide when an excess of lime has been added; and when a deficiency of lime has been added, it may be considered as magnesium hydroxide only. Figure 13 shows this relationship between excess or deficiency of hydroxide alkalinity as obtained by plotting residual magnesium against total hydroxide alkalinity less the hydroxide alkalinity equivalent to the magnesium. For the purpose of calculating chemical feeds, certain residual magnesium may be expected with various excesses or deficiencies of lime, as shown in Fig. 13.

TABLE 10

Plant Operation Comparison

	JAN., 1940	FEB., 1940	MAR., 1940	APR., 1940	MAY, 1940	JUNE, 1940
Total water conditioned, *gal.*	272,506,000	253,102,000	272,643,000	260,875,000	275,894,000	279,412,000
Lime used, *lb.*	784,779	706,781	766,212	796,115	846,456	890,549
Soda Ash used, *lb.*	159,110	73,096	68,904	62,291	70,473	70,587
Coke used, *lb.*	71,490	47,370	55,780	63,750	73,010	63,515
Carbon Dioxide used, *lb.*	—	97,249	91,091	115,389	121,023	91,007
Chemicals: *lb. per mil. gal. treated*						
Lime	2,880	2,806	2,810	2,977	3,068	3,187
Soda Ash	584	228	253	232	255	253
Coke	262	187	205	244	265	227
Carbon Dioxide	—	388	334	434	439	325
Chemicals: *cost per mil. gal. treated*						
Lime at $8.35 per ton	$12.02	11.72	11.73	12.43	12.81	13.30
Soda Ash at $18.612 per ton	5.43	2.68	2.35	2.15	2.37	2.35
Coke at $6.25 per ton	.82	.58	.64	.76	.82	.71
Total	$18.27	14.98	14.72	15.34	16.00	16.36
Ave. hardness water entering, *p.p.m.*	429.0	403.6	418.9	414.2	421.2	419.2
Ave. hardness water leaving, *p.p.m.*	90.0	90.7	87.6	88.0	83.1	79.6
Total hardness removed, *lb.*	770,449	660,495	753,325	850,974	777,953	791,368
Lb. lime per thousand lb. hardness removed	1,018	1,070	1,017	935	1,090	1,125
Lb. Soda Ash per thousand lb. hardness removed	207	111	91	73	91	89
Lb. Coke per thousand lb. hardness removed	93	72	74	75	94	80
Lb. CO_2 generated per lb. coke	—	2.05	1.63	1.81	1.66	1.43
Steam generated by CO_2 boilers, *lb.*	571,920	378,960	369,821	121,275	539,507	470,600
Steam to CO_2 compressors, *lb.*	595,000	695,840	866,136	71,796	0	0
Steam for heat, *lb.*	336,000	407,000	379,000	164,000	82,000	15,000
Kw. hr. used—lights	26,450	18,000	14,180	10,970	10,500	8,400
Kw. hr. used—power	17,000	21,680	25,070	23,220	25,260	22,600

TABLE 11

Average of Average Daily Reaction Records

	FEB. 1940	MAR. 1940	EXPECTED RESULTS
Raw			
Alkalinity—Bicarbonate, *p.p.m.*	343.6	336.5	
pH	7.33	7.39	
Free CO_2, *p.p.m.*	37.8	33.8	
Total hardness, *p.p.m.*	403.5	418.9	
Non-carbonate hardness, *p.p.m.*	60.2	82.6	
Magnesium hardness, *p.p.m.*	39.0	39.0	
Turbidity	8.8	9.8	
Stability (Enslow's method)	—	+7.0	
Aerator			
pH	7.55	7.75	
Free CO_2, *p.p.m.*	22.8	15.7	
Flocculator			
Total alkalinity, *p.p.m.*	80.2	94.5	110
OH alkalinity, *p.p.m.*	40.0	55.6	75
Carbonate alkalinity, *p.p.m.*	40.2	38.6	35
pH	11.01	11.35	11.6
First Settling			
Total alkalinity, *p.p.m.*	66.3	75.9	95
OH alkalinity, *p.p.m.*	26.6	35.2	60
Carbonate alkalinity, *p.p.m.*	39.7	39.1	35
pH	10.77	11.14	11.6
Magnesium, *p.p.m.*	8.8	7.4	1.5
Turbidity	90.2	53.1	10
Carbonation			
Total alkalinity, *p.p.m.*	51.5	51.9	40
OH alkalinity, *p.p.m.*	10.3	7.8	0
Carbonate alkalinity, *p.p.m.*	37.5	34.9	40
Bicarbonate alkalinity, *p.p.m.*	10.0	9.1	0
pH	10.14	10.43	10.6
Turbidity	76.1	57.0	120
Second Settling			
Total alkalinity, *p.p.m.*	49.2	50.2	30
P alkalinity, *p.p.m.*	25.4	26.5	15
pH	10.07	10.25	9.4
Turbidity	45.8	31.7	30

TABLE 11—*Concluded*

	FEB. 1940	MAR. 1940	EXPECTED RESULTS
First Recarbonation			
Total alkalinity, *p.p.m.*	48.1	42.1	30.0
P alkalinity, *p.p.m.*	24.6	20.8	5.0
pH	10.08	10.38	9.0
Filter Effluent			
Total alkalinity, *p.p.m.*	41.4	35.8	25.0
Carbonate alkalinity, *p.p.m.*	15.6	27.2	5.0
Bicarbonate alk., *p.p.m.*	21.3	5.7	0
pH	8.59	10.19	9.0
Final Recarb. Plant Effluent			
pH	8.07	8.38	8.4
Free CO_2, *p.p.m.*	5.7	3.7	2.0
Total hardness, *p.p.m.*	90.7	87.6	85.0
Non-carbonate hard., *p.p.m.*	55.5	51.2	60.0
Turbidity	1.5	0.1	0
Reservoir			
Total alkalinity, *p.p.m.*	36.8	35.6	25.0
OH alkalinity, *p.p.m.*	0.6	0	0
Carbonate alkalinity, *p.p.m.*	6.3	4.5	0
Bicarbonate alk., *p.p.m.*	29.9	31.1	25.0
pH	7.94	8.51	8.4
Free CO_2, *p.p.m.*	4.7	2.3	0.2
Total hardness, *p.p.m.*	91.2	88.1	85.0
Non-carbonate hard., *p.p.m.*	54.4	52.4	60.0
Turbidity	1.0	0	0
Stability (Enslow's method)	—	−4.6	

To prevent complaints due to any possible change in taste, 10 p.p.m. free CO_2 was carried in the finished water during the opening period. This has gradually been reduced and the stability of the water increased. It is probable that the free CO_2 will have to be reduced to zero and the pH increased to 8.6 or above before a stable water is produced. It may even be desirable to supply a water with a still higher pH and a plus stability to stop corrosion and red water when the water is heated.

In the operation of the aerators there is evidently sufficient circulation of air, even without the use of the exhaust fans, so that there is no stratification of the carbon dioxide content in the room. Anal-

ysis shows a CO_2 percentage of less than one, with no greater density at the floor level. The exhaust fans are used only when the weather is heavy or sluggish as is indicated by fogging. Tests to determine the amount of carbon dioxide removal indicate that the residual CO_2 will average from 16 to 20 p.p.m. regardless of the rate at which the water enters the plant and regardless of the fact that influent CO_2 content varies from 30 to 45 p.p.m.

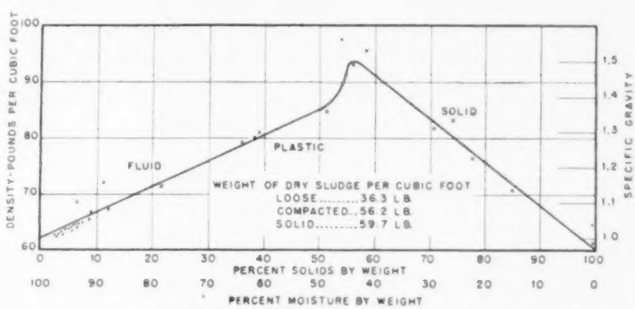

FIG. 15. Density and Specific Gravity of Sludge; as determined from per cent of moisture

TABLE 12

Plant Sludge Analyses

ANALYSIS PERCENTAGE		PROBABLE COMBINATION PERCENTAGE	
CaO	47.32	$CaCO_3$	82.2
MgO	6.53	$Ca(OH)_2$	1.7
CO_2	36.17	$Mg(OH)_2$	9.4
Water & Volatile	7.43	Water & Volatile	4.1
SiO_2	1.65	Silica	1.7
R_2O_3	0.90	Iron	0.9
	100.00		100.0

A centrifuge is used for the rapid determination of solids in the sludge pumped from the plant. The actual percentage of dry solids and the specific gravity of the sludge is then taken from a curve established by experiment (Fig. 14). Attempts to determine the amount of solids by settling showed that no reliance could be placed on this method unless a settling time of 5 hours, and preferably 20 hours, was used.

Figure 15 shows the density and specific gravity of the plant sludge with varying moisture percentages. The transition from fluid to

TABLE 13

Filter Operation

	JAN., 1940	FEB., 1940	MAR., 1940	APR., 1940	MAY, 1940	JUNE, 1940
Total water filtered, *gal.*	276,731,000	257,169,000	270,879,000	265,310,000	280,832,000	284,180,000
Ave. number filters in service per day	5.17	5.17	5.68	5.83	5.58	5.40
Total filter hours	2,097.75	2,174.50	3,049.00	3,095.75	2,823.25	2,440.25
Ave. filter hours per day	67.67	74.98	98.36	103.19	91.07	81.34
Ave. filter hours per day per filter	13.04	14.50	17.30	17.70	16.32	15.06
Ave. filtering rate, *m.g.a.d.*	119.6	104.9	81.14	77.34	91.13	105.77
Ave. filtering rate, *gal. per min. per sq. ft.*	1.91	1.67	1.29	1.23	1.45	1.69
No. filter runs completed	46	44	50	47	53	53
Ave. length of run, *hr.*	74.68	49.40	57.94	69.70	51.44	46.04
*Wash water used, *gal.*	4,325,000	4,067,000	4,295,000	4,435,000	4,938,000	4,768,000
Wash water used, of total water filtered %	1.56	1.58	1.59	1.67	1.76	1.68
Ave. amt. water used per wash, *gal.*	94,021	92,432	85,900	94,362	93,170	89,962
Ave. sand expansion %	30.0	30.28	33.27	35.51	36.94	38.25
Ave. loss of head, initial	1.6	1.15	0.50	0.70	0.81	0.90
Ave. loss of head, final	8.2	7.77	6.18	5.54	6.08	6.43

* All wash water is reclaimed, being pumped to the flocculating basin from a reclaim basin or cistern.

plastic and plastic to solid is not well defined; the general zones only are shown. The sharp break in the curve between 40 and 50 per cent moisture of the sludge covers the range in which the sludge in pondage beds starts to crack and consolidate by shrinkage, changing from the plastic to a solid state.

The pumps and the sludge line are working satisfactorily with the pumpage throttled for sludge having about 10 per cent dry weight solids. Continuous throttled pumpage, rather than periodic pumpage is employed because of the length of the sludge line, a factor which would make necessary considerable waste of clear water to prevent settling in the line at each shut-down. Average sludge analyses are shown in Table 12.

The operation of the filters are best described in tabular form (See Table 13).

TABLE 14

Filter Sludge Analysis

ANALYSIS PERCENTAGE		PROBABLE COMBINED PERCENTAGE	
CaO	52.97	$CaCO_3$	91.4
MgO	1.07	$Ca(OH)_2$	0.4
CO_2	41.37	$Mg(OH)_2$	1.5
Water & Volatile	1.88	Water & Volatile	1.3
SiO_3	2.31	Sand	2.3
R_2O_3	0.40	Iron	0.4
	100.00		100.0

To obtain off-peak electrical rates, pumpage from the wells is doubled during the night. This sudden increase in rate is a major factor in increased turbidity brought onto the filters. The filter sludge analysis is shown in Table 14.

The operation of the chemical-handling, storage and feeding equipment has been dust free and satisfactory. Power consumption for chemical unloading has been 42 kw. per hr., with the operator securing an average unloading rate of 9.2 tons per hr. for lime and 5.8 tons per hr. for soda ash. These rates, the latter especially, will no doubt increase with the increasing experience of the operator.

The plant, consisting of two units in parallel, almost doubles the amount of work from the operator's standpoint; but, even so, the entire plant is being operated by one man per shift of eight hours. Janitor service during the day shift provides the labor for chemical

unloading, and for greasing and oiling. The plant is under the supervision of a chief chemist.

While the operation phase of this plant is still in its infancy, research work has already been started or is projected with relation to the following:

Coagulants: An alum feeder is installed so that experimentation

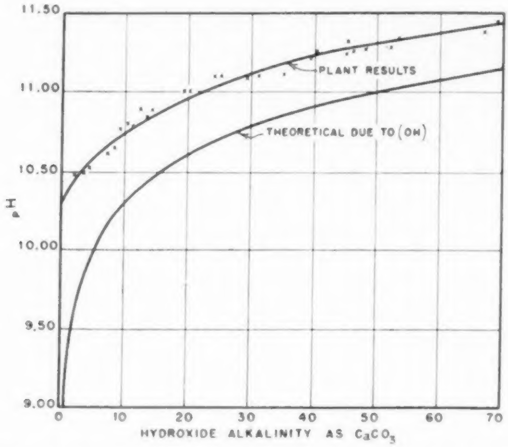

FIG. 16. Relationship between pH and Hydroxide Alkalinity

TABLE 15

Dissolved Oxygen

	p.p.m.
Raw Water	5.79
Aerator Floor	8.62
Aerator Effluent	9.09
Flocculators	9.25
Filter Effluent	8.70
Reservoir, Cold	7.44
Reservoir, Hot	4.72

on a plant scale will determine the advantages if any of using a coagulant. To date, due to the high magnesium content of the water, no added coagulant has been used.

Sodium Hexametaphosphate: Study of the advisability of using sodium hexametaphosphate threshold treatment for prevention of filter sand incrustation and scale in water mains has been initiated.

Sludge Reclamation: Study of carbonation and centrifuging proces-

ses for the removal of magnesium, to permit reburning of the sludge for lime, have been started on a laboratory scale. Studies are also being conducted for sludge by-products.

Dissolved Oxygen Reduction: Study of the possibility of reducing the amount of dissolved oxygen in the plant effluent by physical or chemical means has been started. Table 15 shows the dissolved oxygen content of the water at various points in the plant. It can be seen that the content increases greatly as the water passes through the aerators and flocculating basins. The reduction through the balance of the plant is probably due to the sweeping action of the gases during the carbonating and recarbonating processes. The samples for "Reservoir, Hot" were taken from the electric hot water heater which serves the laboratory.

Stability: Study as to the most feasible method of preventing red water and corrosion in hot water systems is being made.

pH Control: An investigation of the possibility of using automatic pH control of the feeding of chemicals has been initiated. By this method it should be possible to maintain the hydroxide alkalinity of the water in the flocculation basins at a predetermined figure.

In this connection Fig. 16 shows the relationship established between pH and hydroxide alkalinity. The difference between "theoretical" and "plant" results is due, at least in part, to the presence of an hydroxide ion from the hydrolysis of carbonates. This is shown by the fact that plant results show a pH of about 10.3 to 10.4 when only normal carbonate is present.

Tuning Up the New Milwaukee Filtration Plant

By James E. Kerslake

IN PREPARING a paper on the tuning up of a water purification plant shortly after the plant has been put in operation, it is difficult not to discuss in great detail all the adjustments which have been necessary. Should this be done, however, it would undoubtedly give the impression that a great many or unusual number of difficulties were encountered during the tuning up process. It should be emphasized therefore that, when viewed as a whole, with possibly two exceptions, no more than the normal and expected number of corrections and adjustments were necessary in starting up the Milwaukee plant.

An excellent job was done by the designing and construction divisions in the building of the plant. Particularly noteworthy was the flexibility in design which permitted the necessary adjustments to be made without interruption in operation and without impairment to the quality of water being delivered to the consumers. The necessary adjustments were greatly facilitated by the splendid co-operation of the construction division and of the companies which furnished the various types of equipment installed in the plant.

When starting up a new plant, particularly in a place where the only treatment has been chlorination alone, it is first necessary to hire and to train an operating force. All men employed at the Milwaukee plant, including the superintendent, obtained their positions through civil service examinations. The City Service Commission supplied lists of results from which appointments were made in order of the standings obtained in the examinations.

A paper presented on May 23, 1940, at the Illinois Section Meeting in Chicago, by James E. Kerslake, Superintendent of Filtration, Water Purification Plant, Milwaukee, Wis.

Personnel and Work Schedule

A total of 52 permanent and 5 part-time employees now constitutes the staff. The positions held, together with the work schedule followed, is shown in Table 1.

TABLE 1

Milwaukee Water Purification Plant Organization and Time Schedule

SHIFTS*	PUMP OPERATORS	ASST. PUMP OPERATORS	FILTER OPERATORS	FILTER HELPERS	CHEMICAL FEED OPERATORS	CHEMISTS	ADDITIONAL PERSONNEL
Morning—7:00 A.M. to 3:00 P.M.†	1	1	1	1	1	1	0
Afternoon—3:00 P.M. to 11:00 P.M.†	1	1	1	1	1	1**	0
First Swing‡	1	1	1	1	1	1	0
Night Shift—11:00 P.M. to 7:00 A.M.#	1	1	1	1	1	1**	0
Second Swing§	1	1	1	1	1	1	
Shop—8:00 A.M. to 4:30 P.M.	1	1	2¶	1	2¶	3††	12‡‡

Summary of Personnel: *Full Time:* 1 superintendent, 1 assistant superintendent, 1 clerk, 6 pump operators, 6 assistant pump operators, 16 filter and chemical feed operators, 6 filter helpers, 2 machinists, 6 chemists, 6 laborers, 1 electrician; total, 52; *Part Time:* 1 electrician—half-time, 4 laborers—6 months work on grounds; total, 5.

* Shifts rotate every four weeks.

† Mondays and Tuesdays off.

‡ Mondays and Tuesdays on Morning Shift and Wednesdays through Fridays on Shop Shift, with Saturdays and Sundays off.

\# Wednesdays and Thursdays off.

§ Mondays and Tuesdays on Afternoon Shift and Wednesdays and Thursdays on Night Shift, with Fridays as sleep days and Saturdays and Sundays off.

¶ One filter operator and one chemical feed operator assigned to permanent shop shift. Have had special instruction by company representatives to qualify as experts so that they can supervise repair work.

** Filter and chemical feed operators assigned to laboratory work.

†† One chief chemist, one senior chemist and one bacteriologist.

‡‡ One superintendent, 1 assistant superintendent, 1 clerk, 2 machinists, 6 laborers and 1 electrician.

The schedule has been arranged in accordance with the practice at the two steam pumping stations. It should be noted that there are six shifts with six men on each shift. Four of the men are at

fixed posts, leaving two available to clean and to render any assistance required if unusual operating difficulties are encountered. Twenty-four-hour laboratory control has been established by assigning two of the chemically-trained filter and chemical feed operators to work in the laboratory. It is felt that it is better to do more of the routine cleaning during the night when there are no visitors and when no work is being done by the shop crew.

All men work 40 hours a week except the second-swing shift which works 32 hours. The shifts are changed every four weeks. The 32-hour week which the men get for four weeks twice a year compensates them for the loss of holidays. Men on the shop shift do not work on holidays. Having six shifts permits $1\frac{3}{5}$ shift to be assigned to shop duty when the men are not taking the place of others who are sick or on vacation. This arrangement also makes available trained operators to do the maintenance work. It has worked out as well at the filter plant as it has for years at the two steam pumping stations. Depending on the operators to do maintenance work in their spare time is not, in the author's opinion, good water works practice.

It will be noted that there is only one clerk. His time is pretty well taken up with bookkeeping, and the operators therefore do most of the calculating and keeping of records. By using good paper and a 3-H drawing pencil, the necessity for typewriting the daily records has been eliminated. Typewritten reports of the monthly summaries are prepared.

Additional help needed can also be obtained from the two pumping stations which have larger machine shops than that of the filter plant. The assistant chief engineer of power plants spent about three-fourths of his time at the filter plant during the tuning-up period, and the repair work has been directly under his supervision.

To train the operating personnel a skeleton crew was picked from the top men on the civil service lists of November 1, 1938. These men assisted the construction division in calibrating the chemical feed machines, checking the rate controllers, etc. A full crew was hired on May 16, 1939, and the plant was operated at lake level with the bypass open for six weeks prior to closing the bypass gate. During this period, the plant was first run on eight-hour shifts with all men present, then two shifts per day, and finally three shifts per day, until all were familiar with their duties.

In order to make the men more familiar with water works prac-

tice, the following low cost library was obtained and made available to the men: "Water Supply and Control" by Charles R. Cox, Bulletin No. 22 of the N. Y. State Dept. of Health; "Water Purification for the Practical Man" by Charles R. Cox; "Water Supply and Treatment" by Charles P. Hoover, Bulletin No. 211 of the National Lime Assn.; "The Operation of Water Treatment Plants" by W. A. Hardenbergh and consulting staff, an article in the April, 1938 issue of *Public Works;* the annual data section published by *Water Works and Sewage;* and "Copper Sulfate in Control of Micro-organisms" by Dr. Hale, issued by the Nichols Copper Company.

For general reading a copy of the manual for water works operators prepared by the Texas Water Works Short School staff, and the latest manual of the A. W. W. A. on water quality and treatment were obtained. There are, of course, many other trade publications available as well as other books, but this list contains those most readily available to the staff.

Before discussing the adjustments that were made to the equipment, it would be natural for one to ask, after noting the number and type of men employed, what is the operating and maintenance cost. The estimated cost of operation and maintenance on the delivery of 35 billion gallons per year to the pumping stations is $250,000. The approved budget for 1940 is $236,000, exclusive of heating. The cost of steam for heating furnished by the North Point Pumping Station is approximately $10,000 a year, all of which makes our budget approximately $4,000 less than the original estimate. The cost per million gallons based on the original estimate is $7.14. The actual cost will probably be lower than that, depending on the amount of activated carbon required. Our present estimate is somewhat between $6.50 and $7.00 per million gallons. This cost includes maintenance. The cost for power and light is about $1.45 per million gallons, and the cost of maintenance, including labor and materials, is approximately $1.10 per million gallons.

Changes and Adjustments Made

The necessary work in tuning up the plant can be divided roughly into the following three divisions: (1) additions other than provided in the original plans; (2) replacements of existing equipment; and (3) corrections and adjustments necessary to put the existing equipment in proper working order.

The additions to date other than provided in the original plan are as follows: (1) installed two 36-inch wash water valves in the pipe gallery so that the wash water could be cut off individually from each quarter; (2) placed Dresser couplings for the insertion of a cut-off plate on the discharge line of all pumps so that each cone valve could be isolated individually (otherwise it would have been necessary to cut out an entire row of pumps when making repairs to the cone valve); and (3) installed glass block windows in the filter building to reduce condensation.

The following existing equipment was replaced: (1) two plunger type sampling pumps were replaced by centrifugal pumps as it was found that the plunger type would not stand up under continuous operation unless they were equipped with air chambers which would prevent the collection of a representative sample; and (2) recently developed snubbers were installed on the pneumatic conveyors both at the plant and at the unloading station following complaints of noise from people living in the vicinity.

The following corrections and adjustments were made to existing equipment:

1. *Meters:* Before the meters would operate properly, a complete check of the wiring had to be made. When making the original installation the electrician cross-connected the two supply systems in the two loop supply circuits. Because of the collection of air in the Venturi tube piezometer rings, float-operated air-vent valves were also installed. The air caused faulty differentials to be transmitted to the meters.

2. *Filter Valves:* Larger guide rails and new discs were installed on all 36-inch wash-water valves. The 20-inch surface-wash valves were sent back to the factory for repairs similar to the wash-water valves. The lower discs on the two-disc effluent valves were replaced and the rails adjusted so that a better seat was obtained.

3. *Filter Rate Controllers:* An automatic shut-off valve was installed in the pressure line from the Venturi throat to the under side of the diaphragm which operated to close this line with the action of the effluent valve. This prevented the controller diaphragms from being reversed, which, in consequence, caused certain parts to become jammed and bent. Considerable study, plus the taking of actual photo records showing the pressure under various operating conditions, was necessary before the correction to prevent this overturning of the diaphragms could be made. It is interesting to note

that no such reversal was obtained in an 8-inch model tested in a hydraulic laboratory prior to the installation of controls of this type at the Milwaukee plant. It was also necessary to repack some of the guillotine valve spindles due to sticking. As shipped from the factory, the packing gland was hermetically sealed and it was necessary to make a cut with a special tool in order to insert packing rings. Once the packing rings are inserted, it is a very simple matter to repack the valves as needed.

4. *Flocculators:* All of the gears on the flocculators after four months of operation showed excessive wear on the gear and pinion. This was due primarily to the failure of the two vertical shaft bearings and the horizontal bearing nearest the gear. The material used in the bearings (layers of canvas impregnated with a hardened plastic) was badly worn. These bearings were replaced with a lubricated bronze sleeve type of bearing. The misalignment of the shafts was corrected and new pinions and gears were installed. The new gears are made of a different material containing molybdenum and is specially heat treated. The advantage of this material is that it is much harder and can be cast with a minimum amount of warping. All material was furnished by the original contractor and the work was done under his supervision.

5. *Pneumatic Conveying Equipment:* This equipment both at the plant and unloading station has worked satisfactorily except for the muffler replacements noted above. Nothing except minor adjustments has been required.

6. *Chlorinators:* These proved to be satisfactory, the only trouble being with the collection of gum in the selector valve on the control panel adjacent to the ton containers. Up to date this trouble has been held to a minimum by periodical cleaning. It is being given further study in cooperation with the manufacturer of the equipment.

7. *Surface-Wash System:* A few of the Dresser couplings have worked loose in the header piping. This action has been corrected by tightening with a special wrench furnished by the manufacturer. Movement of the pipe has been prevented by placing wood blocking between the ends of the pipe and the wall of the filters.

8. *Dry Chemical Feed Equipment:* A screw type removable conveyor was added to the lime machine to convey the material from the feed belt to the drum. The sprays in the drum were rearranged to keep the drum clean. Some of the sprays were connected directly

to the hot water system to increase the temperature of the mixture in the drum. Better operation would probably have been obtained if larger doses had been required, and if the machine had been operated nearer to its maximum capacity. The alum machines are equipped with a stainless steel dissolving tank and little difficulty has been experienced in their operation. The ammonium sulfate machines, too, have given but little difficulty since they were repainted with acid resisting paint.

Little trouble has been experienced with the carbon machines when feeding at low capacity. This does not hold true, however, when the machines are run near the maximum capacity. Further studies are to be made and it may be necessary to put in larger vortex bowls and additional suction equipment for removing dust. A model of the carbon bag dump with the opening closer to the floor and with the reshaping of the dust hood to get a more streamlined effect has been built and found satisfactory. The rebuilding of the present carbon bag dumps to conform with the model is contemplated.

In the chemical feed room additional drains are being considered to remove any water collected on the floor at a faster rate.

The storage of chemicals in quantity in steel bunkers has proved satisfactory for all chemicals except carbon. Recently a fire in the carbon bunker had to be extinguished with carbon dioxide gas. To combat such fires the following recommendations have been contemplated:

a. Keeping on hand an additional 200 lb. of carbon dioxide gas for use in the event of other fires in the carbon bunker.

b. Equipping the bunkers with explosion-proof doors, similar to those used on grain elevators.

c. Placing indicating thermometers in each bunker.

d. Purchase of more metal cans for the storage of empty carbon bags in which to transport the bags to the incinerator.

e. Installation of automatic equipment which would flood the bunker with carbon dioxide when the temperature rose to a certain fixed point.

In addition to the repairs and replacements of equipment as outlined above, some trouble has been experienced with ice during the winter when a strong east wind packs slush ice into the wash-water drain line. So far this has been corrected by building up the head on

the lines during every wash, and blowing them out. Further study of the problem is being made.

9. *Baffles:* As recommended by the contractor furnishing the mixing equipment, the baffles in the mixing basin were lowered 7 ft., decreasing the clearance from 10 to 3 ft. It is believed that this has helped to prevent short circuiting, and also that a better formed and more rapidly settling "floc" has been obtained since the change was made. This work was carried out at the same time that the flocculators were being repaired.

Cleaning and Floor Maintenance

In keeping a plant of this size clean, the work is greatly facilitated by obtaining some of the more recently developed scrubbing, polishing, and suction machines. The suction machine has been found particularly valuable in picking up carbon which has previously been wetted down, and in cleaning the chemical feed machines to prevent the collection of dust which settles out in the cabinets if the machines are permitted to go too long without cleaning.

Considerable study has been made of the proper treatment of the floors in the plant, and a scrubbing and waxing program has been worked out. The carrying out of this program has been greatly facilitated by the purchase of the polishing and scrubbing machine mentioned.

Large Numbers of Visitors

Since the beginning of the year there have been from 1,800 to 5,500 visitors per month. It has been necessary to set definite visiting hours of from 2 to 5 P.M. daily. There is a lecture room and a projection machine and a special tour with a lecture is given to groups provided arrangements have been made beforehand. Groups are permitted to visit the plant at night from Monday through Thursday provided the necessary arrangements have been made. No provision was made in the budget for guide service, and a considerable portion of the supervising staff's time and operator's time has been taken up by visitors.

Each man at the plant was required to provide himself with three uniforms when reporting for duty. A coverall type of uniform was selected, and to start off the program, the uniforms were rented from a laundry company so that all uniforms would be alike. A charge for such service was $0.35 per garment per week. A credit of $0.10

was allowed toward purchasing the garment each time they were laundered. Practically all the men in the plant have purchased their own uniforms and now have them laundered independently. Insignia for the uniforms were furnished by the water department. The costs of the uniforms were as follows: coveralls, \$2.65 each; caps, \$1.50 each; and cap covers \$0.50 each.

When taking bids for chemicals, prices f.o.b. the freight unloading station as well as truck delivery at the plant were obtained. In a number of instances, it was found more economical for the city to purchase chemicals delivered at the plant by truck, the bidder quoting a lower price for plant delivery than the city can purchase by freight delivery and do its own hauling.

Operating Experience

Little difficulty was experienced in obtaining a safe, clear water free of tastes and odors, once the equipment was finally adjusted to operate properly. The main problem has been one of taste and odor removal. In order to produce a final effluent with a threshold odor below three, and at the same time to keep carbon cost down, it has been necessary to run hourly threshold odor tests on the raw water. It has been found that practically all the odors are due to algae growths, and that the odor in the raw water rises very rapidly with the change in direction and velocity of the wind. This makes it necessary to apply a large amount of carbon at rapidly increasing rates to produce a satisfactory effluent. Serious consideration is, therefore, being given the installation of additional carbon bag dumps so that, if necessary, carbon can be applied by hand to the raw water during periods of high threshold odors. These periods would be of short duration. The present total capacity of the carbon machines is about 1,000 lb. per hr., or 24,000 lb. per day, and for the short time that there would be doses in excess of this amount, it would not be economical to purchase additional carbon feed equipment.

A sight well has been provided in the filter building. It has proved very interesting to the visitors and has been of great aid to the chemists in detecting any slight increase in the turbidity. In fact, any turbidity in the effluent will be noticed in the sight well before it can be detected in the laboratory. It has been endeavored to produce a water of zero turbidity, i.e. less than 0.05 parts per million.

Plant Tests

The plant is so designed that it can be readily divided into two equal sections. This enables the running of tests on a plant scale. Such tests have been carried out in comparing four different brands of carbon, and preparations are being made to start tests comparing ferric sulfate with alum. Laboratory tests are run first and, after they are tried out in the laboratory, plant scale tests are run, provided, of course, that the laboratory tests show it can be done without impairing the quality of the plant effluent. These tests will be continued in the future. At present there is particular interest in break-point chlorination. It is felt that much more conclusive results can be obtained in treating 20 or more million gallons in place of treating a small quantity in the laboratory.

Complaints

When the plant was first started, a number of complaints due to turbidity, taste and odor, chlorine taste, killing of fish, and the increase of temperature were received. In some instances, these complaints were due to the feeling, on the part of the public, that the results obtained had not justified the large expenditure incurred in building the filter plant. The complaints are becoming fewer and fewer, however, and it is hoped they will cease eventually. Occasional complaints of turbidity are to be expected until the bulk of the deposits in the distribution system, placed there prior to the use of the plant, have been flushed out. The filter plant was blamed for a rise in the temperature of the water but the real cause was the nature of the season. The data indicate that the variation between the raw and filtered water is less than one degree. There is, however, quite a change in temperature during the summer season with the change in direction and velocity of the wind. Except for a short time after the plant was first started, when it was not possible to apply sufficient carbon to remove the taste and odor, complaints as to taste and odor have virtually ceased. The same is true of chlorine tastes. Complaints as to tastes were probably accentuated at first, with the introduction of the ammonia-chlorine treatment and, now that water containing a residual chlorine content has been distributed to all parts of the distribution system for a period of more than six months, complaints from this source should be eliminated.

References from literature of the field regarding the failure of the equipment or the installation of adequate equipment have been noted from time to time. It is the author's belief that complaints as to failures of equipment can be eliminated if the operating staff, the construction division, and companies who furnish the equipment will cooperate wholeheartedly in correcting and adjusting the equipment which has been installed.

The bulk of operating difficulties during the tuning-up period in Milwaukee can be traced directly to the large size of the units installed where previously operating experience was limited to a small number of plants, many of which were constructed prior to the development of the equipment in use at Milwaukee.

Illustrated pamphlets describing the plant have been printed and are distributed to visitors who are particularly interested. It has been found that these pamphlets are particularly helpful to school groups as an adjunct to the lecture.

Improvements to Wichita Water Supply

By R. E. Lawrence

THE new water supply system for the City of Wichita, Kansas consists of the following major items of construction:

(1) The testing and development of a new ground water supply of approximately 32 million gallons per day.

(2) The construction of gathering lines from the 25 wells in the supply field and a supply main from these gathering lines to the water filtration plant in Wichita.

(3) The construction of a water filtration plant with a nominal capacity of 32 m.g.d., including a total filtered water storage of 4 million gallons. This plant is located in Wichita near the plant of the privately owned Wichita Water Company.

The City is now served by the Wichita Water Company with a supply taken from a group of shallow wells penetrating the underflow of the Arkansas and Little Arkansas River at Wichita. The supply is adequate in quantity and is cheaply produced. It is, however, highly mineralized and contains excessive quantities of chlorides which render it undesirable for domestic and industrial use.

The new supply is obtained from a group of 25 wells located in the general area west and south of the City of Halstead and is secured from what is locally called the Equus Beds Area. This geologic formation, covering portions of McPherson, Marion, Harvey, Reno, and Sedgwick Counties, is known to be a remarkable aquifer and has been utilized by a number of municipalities as a source of water supply. In general, the strata that compose the Equus Beds consist of alternating layers of sand and clay. The depth to shale in that portion of the area from which the new Wichita supply is secured, varies from 150 to 250 ft. and the static ground water level is within a few feet of the surface of the ground.

A paper presented on April 23, 1940, at the Kansas City Convention by R. E. Lawrence, Black & Veatch, Consulting Engineers, Kansas City, Mo.

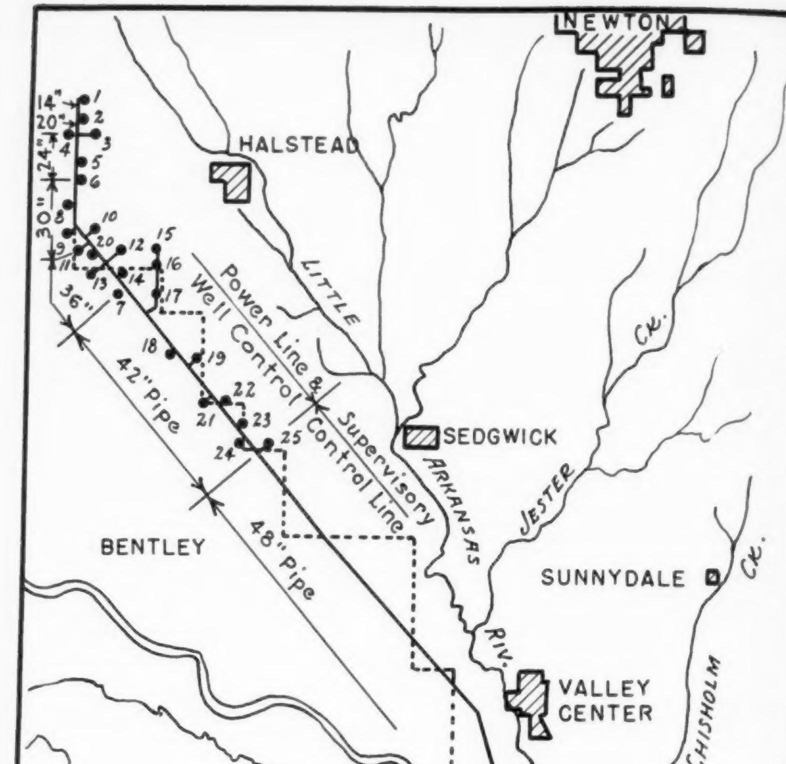

FIG. 1. Wells, Pipe Lines, Route of Supervisory Control and Power Lines—Wichita Water Supply System

The 25 water supply wells are of the gravel wall type. The diameter of the hole at the bottom of the well is 30 in. and the diameter of the inside casing is 18 in. The shallow wells were constructed by the casing method and the deeper ones by rotary drilling.

In order to prevent a sharp depression of the ground water table in any particular locality, a minimum spacing between wells of ½ mi. was maintained and the wells as located intercept a front of approximately 9 mi. perpendicular to the direction of ground water flow.

Twenty-two of the wells are being equipped with 1,000 gallon per minute vertical turbine type pumps and three with 500 g.p.m. pumps. The well pumps discharge through spur lines direct into the supply pipe line which conveys the water to the new filtration

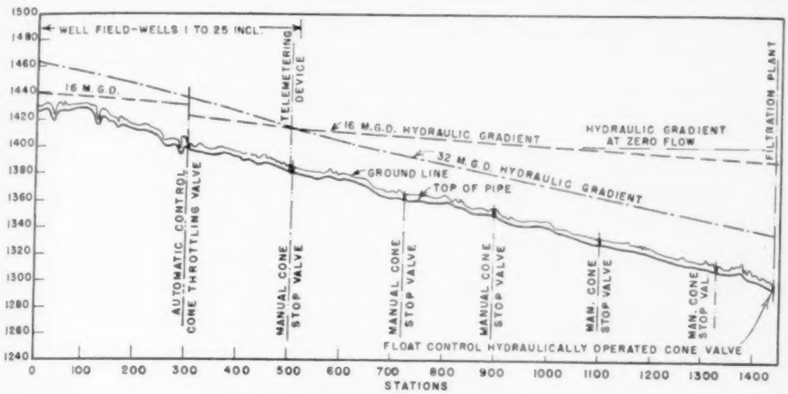

FIG. 2. Profile of Pipe Line

plant at Wichita. The location of the well field and the route of the pipe line are shown on Fig. 1.

The pipe line contains, in addition to 25,400 linear ft. of 14-inch cast iron pipe in gathering lines, the following quantities and sizes of cast iron pipe:

48-inch	93,100	lin. ft.
42-inch	27,200	"
36-inch	8,200	"
30-inch	7,300	"
24-inch	7,800	"
20-inch	5,300	"
Total	148,900	lin. ft.

As may be seen from Fig. 2, the profile of the pipe line has a relatively uniform slope over its entire length, with a total drop in eleva-

tion from the upper end of the well field to the filter plant of approximately 130 ft. The normal line discharge, however, will be through the plant aerator, which reduces the available static head differential over the entire length of the line to approximately 90 ft. This head is adequate for handling all normal demands by gravity flow through the line, but some help will be required from the well pumps for handling maximum demands, and to supply the maximum designed capacity of the filter plant will require an increase in pumping heads at the wells varying from approximately 40 ft. at the upper end of the well field to 30 ft. at the lower end.

Pipe line friction was computed by the Williams and Hazen formula, using a friction coefficient of 140 which is believed to be a conservative figure for the cement-lined pipe being installed.

In order to prevent entrance of air into the pipe line through automatic air vent and vacuum-breaking valves, the entire line will be kept full of water at all times, regardless of the filter plant demand and the slope of the hydraulic gradient, by means of two automatically operated and controlled regulating valves of the cone type. One of these valves is located at Station 298+00 and the other at the filtration plant in Wichita. The automatic cone valve at the filtration plant is provided with dual controls, a pressure control for maintenance of a minimum pressure above the valve to prevent access of air into the pipe line between the plant and the other control valve at Station 298+00, and a float-operated control which, operating from fluctuations of water level on the filters, will regulate the flow through the pipe line to conform with the demands of the filters.

Maximum internal hydrostatic pressures would occur in the pipe line if the regulating valve at the filtration plant were completely closed and one or more pumps in the upper portion of the well field remained in operation. It is not contemplated that this condition will occur in normal operation, but to reduce the combined static and pump shut-off pressures to a minimum, the well pumps have been designed with shut-off pressures as low as practicable. Both the upper and the lower sections of the pipe line system are protected from pressures in excess of design assumptions by pressure relief valves placed at the lower end of each section.

The design of the cast iron pipe used in this pipe line was based on the principles and methods described in Part 1 of the *Manual for the Computation of Strength and Thickness of Cast Iron Pipe* (1), which was approved by the American Standards Association in December,

1939. This new theory and method of design was developed by Sectional Committee A21 of the American Standards Association from experience and data obtained through considerable research on the properties of cast iron and extensive tests made on cast iron pipe subjected to the simultaneous application of external loads and internal pressures, and has been fully presented in the excellent paper by Wiggin, Enger and Schlick (2).

Application of New Design Method

In addition to static pressure, water hammer, foundry tolerance, and corrosion factor, design computations made in conformity with this new method include the external loads imposed by earth pressure and trucks and the effect on such external loads and resulting stresses in the pipe barrel of different methods of pipe laying and bedding, the tamping or lack of tamping of backfill, and the depth of earth cover over the pipe.

This new method enables the engineer to design and, through its acceptance by the pipe manufacturers, to obtain cast iron pipe of an adequate, but not excessive, thickness to withstand any condition of internal pressure and external loading, and in addition to take advantage of any available strength and quality of iron. Instead of being restricted to the few standard classifications available under the older standard pipe specifications, the engineer can now design pipe of as many thicknesses as may be required to meet varying conditions of pressure and external loads. The old methods of pipe classification, being based on internal working pressure, required that all pipe within the class pressure limits be of the thickness necessary for the maximum pressure condition encountered even though most of the line might be subjected to a much lower pressure. The older methods of pipe design also assumed external loadings which were based on ordinary distribution system service and which were not only excessive for shallow cross-country transmission lines but entirely inadequate for large diameter pipe subjected to low internal pressure and heavy superimposed loads. Inasmuch as external loadings did not appear in the pipe classification and barrel thickness tables, the average pipe line designer had no way of knowing just what external load the pipe would stand. In the case of pipe larger than 24 inches in size which was designed for a low working pressure, the external loads would often govern the pipe barrel thickness, and the determination of the proper pressure classification necessary to

withstand heavy superimposed loads, if any such determination was made at all, had to be based on either rather uncertain ring formulas or on the engineer's experience and judgment. The usual practice of many designers and specification writers has been to ignore external loads entirely, a procedure satisfactory enough for small diameter pipe, but rather dangerous for large pipe.

In specifying and designating the Wichita pipe a number of special pipe classifications based on both internal working pressures and external loadings, were used. The pipe was classified for internal working pressures as Class 50, Class 75, and Class 100, the figure in each case representing the water working pressure of the pipe in pounds per square inch. This is the system of pressure classification used in Federal Specifications WW-P-421. The pipe was classified for external dead loads as "Standard," Class 3.5 and Class 5.0. The "Standard" class, which carried no classification symbol, was used

TABLE 1

Maximum Depths of Cover for Various Classifications of Pipe

DEAD LOAD CLASSIFICATION DESIGNATION	MAXIMUM DISTANCE OF TOP OF PIPE BARREL BELOW:	
	Original Ground Surface	Finished Top of Backfill
	ft.	*ft.*
None (Standard)	$2\frac{3}{4}$	$3\frac{1}{4}$
3.5	4	$4\frac{3}{4}$
5.0	$5\frac{3}{4}$	$6\frac{3}{4}$

for a nominal depth of cover of 2.5 feet or less. The figures in the other two classes represent the nominal depth in feet of the earth-cover over the pipe. The dead-load classifications and permissible tolerances for the three classified depths are shown in Table 1.

Dead loads were based on Field Condition "B," as defined in Part 1 of the design manual previously mentioned. The definition assumes a flat bottom trench and tamped backfill. The trenching and pipe-laying specifications were written to insure this condition. Live loads were divided into two classes, "Standard" and "Highway," which were based on location of the pipe. The standard live-load classification, which carried no symbol, included all pipe laid across fields and open country outside of highways, streets, and railroad rights-of-way, except that in certain locations where the depth of cover exceeded 5 ft., the highway classification was used to avoid an additional classification for a very small amount of pipe.

The live load used in the standard live-load classification, representing cross-country conditions outside of highways, was that imposed by a 6,000-pound wheel load which, with 80 per cent rear axle load distribution, would be that imparted by a 5-ton truck with a 50 per cent allowance for impact or a $7\frac{1}{2}$-ton truck with no impact allowance. The assumption on which these two truck sizes were used takes into consideration the probability that a truck weighing more than 5 tons would be traveling so slowly across cultivated fields that the wheel impact would be negligible.

The live load used for the *X* or highway classification was that imparted by a 15-ton truck with 80 per cent rear axle distribution and a 12,000-pound rear-wheel load, plus a 50 per cent allowance for impact. This loading is one-third heavier than the 9,000-pound wheel load described in the "Manual," but was considered advisable because of the heavy trucking incident to local oil field operations. This live load was assumed as acting through a depth of backfill of 6 ft. over the pipe, this representing the greatest depth of cover over pipe located in highways on the project. The depth also requires a slightly greater pipe barrel thickness than any lesser depth. The difference in thickness requirements for the highway truck loads acting through depths of cover ranging from 2.5 to 6 ft. were so small that it was not considered desirable to provide different depth classifications for the small amount of *X* pipe required for the work.

A factor of safety of 2.5 was applied to all internal pressures and external loads, as assumed by Committee A21 and stipulated in Section 1-10 of the *Manual.*

The *Manual* stipulates further that water hammer and the load and impact from passing trucks are not assumed as acting simultaneously, but that both are to be considered separately as loads to be added to static pressure and backfill load, and that whichever of the two gives the greater pipe thickness will be the controlling factor in the pipe-thickness determination. Because of the numerous precautions taken to reduce the effect of water hammer in the line, a factor which will be referred to later in this paper, a design allowance of 20 lb. per sq.in. was made. This allowance lacked a considerable amount of equaling the effect of the assumed truck loads on the larger pipe, and in all classes and sizes of pipe larger than 24 in. in diameter the controlling factors were internal pressure and the combined dead and live loads. For this reason no water hammer allowance was used in the calculations for pipe of these sizes.

A foundry tolerance of 0.10 in. for all pipe larger than 24 in. in nominal internal diameter and 0.08 in. for pipe smaller than 24 in. in diameter, and a corrosion allowance of 0.08 in. for all sizes, was added to the calculated thickness for each size, class, and quality specified.

Pipe barrel thicknesses were calculated and specified for all of the designated classes for pipe made from each of five different "qualities" of iron. This was done to enable pipe manufacturers to select the quality of iron, within the limits of the five qualities specified, which they could best produce; and to let them manufacture pipe, from iron of such selected quality, of definite specified barrel thicknesses. The stipulation of definite pipe thicknesses for several qualities of

TABLE 2

Test Requirements in Pounds per Square Inch for Various Qualities of Pipe

	QUALITY A	QUALITY B	QUALITY C	QUALITY D	QUALITY E
Tensile Strength	16,000	18,000	21,000	23,000	25,000
Modulus of Rupture	40,000	40,000	45,000	48,000	50,000
Modulus of Elasticity	10,000,000	10,000,000	11,500,000	12,100,000	12,500,000

iron also permitted the manufacturer to change, on a definite specification basis, the quality of iron produced, if it was found difficult or uneconomical to maintain the selected quality. Also, if desirable, it made it possible for him to reduce the weight of the pipe by producing iron of a higher quality.

The several grades of iron were designated as "Quality A, B, C, D and E." The test requirements in pounds per square inch for the several qualities of iron and pipe made therefrom were as shown in Table 2.

The tensile strengths and moduli of rupture given in Table 2 are minimum requirements. The moduli of elasticity are maximum requirements. In case the modulus of elasticity should exceed the stipulated maximum for any quality, the minimum modulus of rupture for that quality was required to be increased by at least the same percentage.

The minimum thickness in inches, including foundry tolerance and corrosion allowance, for each designated size and classification of cast iron pipe was specified to be as shown in Table 3.

The cast iron pipe for the project is being furnished by the Ameri-

can Cast Iron Pipe Company, which is supplying centrifugally cast cement-lined pipe. The 48-inch pipe is the largest centrifugally cast pipe yet to be manufactured, and the first pipe of this size to be produced under the "New Law of Design."

TABLE 3

Minimum Thicknesses for Pipe of Various Diameters and Classifications

PIPE SIZE	PIPE CLASS	MINIMUM THICKNESS IN INCHES				
		Quality A	Quality B	Quality C	Quality D	Quality E
in.						
30	50	0.64	0.63	0.61	0.60	0.59
30	50-3.5	0.70	0.69	0.66	0.64	0.63
30	50-5.0	0.73	0.72	0.68	0.66	0.65
30	50×	0.80	0.79	0.75	0.73	0.72
36	50	0.71	0.70	0.67	0.65	0.64
36	50-3.5	0.77	0.76	0.73	0.71	0.69
36	50-5.0	0.81	0.80	0.77	0.74	0.73
36	50×	0.91	0.90	0.86	0.83	0.82
42	50	0.76	0.75	0.71	0.69	0.68
42	50-3.5	0.80	0.79	0.75	0.73	0.72
42	50-5.0	0.88	0.87	0.83	0.81	0.79
42	50×	0.99	0.98	0.94	0.92	0.90
48	50	0.82	0.81	0.77	0.75	0.74
48	50-3.5	0.87	0.86	0.82	0.80	0.78
48	50-5.0	0.96	0.95	0.90	0.88	0.86
48	50×	1.09	1.08	1.03	1.01	0.98
48	75	0.85	0.83	0.79	0.76	0.75
48	75-3.5	0.90	0.89	0.84	0.81	0.80
48	75-5.0	0.99	0.98	0.93	0.90	0.88
48	75×	1.12	1.11	1.05	1.02	1.00
48	100	0.89	0.87	0.82	0.80	0.78
48	100-3.5	0.93	0.92	0.86	0.84	0.82
48	100-5.0	1.02	1.01	0.95	0.92	0.90
48	100×	1.14	1.13	1.07	1.04	1.02

For most economical design it was necessary to reduce the effect of water hammer to such an extent that the controlling factor in each pipe thickness calculation should be the assumed external live

load. This required the determination not only of practical means for reduction of line surges and water hammer but of the intensity of pressure waves resulting from operation of the protective devices provided by the design. Because of the great length of the line, approximately 28 miles exclusive of the branch lines to the wells, and the necessity for controlling rates of flow to provide for highly variable demands, the problem of water hammer control was somewhat complicated.

After making a study of various methods of water hammer analysis, it was finally decided to use an analytical method developed by Gibson (3). This analysis is based on the theory of pressure waves and consists of a method of tracing the action of such waves instant by instant. The accuracy of the Gibson method has been verified experimentally and can be relied upon for design purposes.

Valves vary greatly in their characteristics with respect to water velocities during closure. An ordinary gate valve may produce velocities varying considerably from the initial velocity at beginning of closure to the maximum velocity near final closure. It is therefore desirable, for water hammer control, to select a type of valve that will have a minimum variation of water velocity change during the closure operation. Cone valves are well adapted for this type of service since the rate of closure can be adjusted to equalize the rate of change of water velocity. Detailed calculations were made based upon various arrangements of valves and assumed conditions of operation. As a result of these studies it was concluded that the use of cone valves would make it possible to control water hammer in the pipe line to within 20 per cent of the static pressures.

It was decided to use automatic hydraulically operated cone valves on each well-discharge line and a float-controlled hydraulically operated cone valve at the filter plant. Also pressure relief valves were provided in the lower end of the main line at the filter plant and near the upper end of the line to flatten out unforeseen pressure peaks. Owing to the fact that the 25 well-discharge lines connect with the main supply line throughout the full length of the well field, any analysis taking into consideration the effect of closing valves in the discharge lines from the pumps, either in sequence or simultaneously, could be only an approximation. By providing valves at these installations which can be adjusted for time of closure, however, it will be possible to make such final adjustments as will be required after the system is in operation.

Valve System

The valve layout for the entire pipe line consists of the following:

(1) A float-controlled hydraulically operated cone valve on the lower end of the line at the filter plant.

(2) Four 30-inch and one 24-inch manually operated cone valves located at intervals of about five miles in the pipe line.

(3) One 20-inch automatically controlled and operated cone valve in the pipe line at Station 298+00.

(4) One group of 6-inch angle-needle type pressure relief valves to be installed in the lower end of the 48-inch supply line at the filtration plant, and one group of four 4-inch angle-needle type pressure relief valves to be installed at Station 285+00 on the pipe line.

(5) Twenty-two 8-inch and three 6-inch automatically controlled and operated cone check valves on the discharge lines from the well pumps.

(6) Air release, vacuum release, and manually operated blow-off valves at selected points along the pipe line.

(7) Gate valves on all branch well lines at the point of connection with the main pipe line.

Position indicators showing the position of the plug are furnished with each cone valve and the rotating levers are so proportioned as to cause the plug to cut off not less than 75 per cent of the port area with 40 per cent of the piston stroke.

The purpose of the main outlet valve at the filter plant is two-fold. First, it is to keep that portion of the pipe line between the filter plant and the upper control structure full of water at all times regardless of the demand or slope of the hydraulic gradient. Second, it regulates the output of the pipe line to conform with the demand requirements of whatever filters may be in operation. The main control for the outlet-throttling valve operating the hydraulic cylinder will be the water level in a basin connected to and fluctuating with, the filters. The main outlet valve will close as the basin water level rises and will open as the basin level falls.

The throttling valve at Station 298+00 will keep the pipe line above this point full of water at all times, regardless of pumping operations. At no flow, the valve is to be entirely closed and will assume this position at any time the hydraulic gradient at the valve drops to elevation 1430. At maximum flow the hydraulic gradient at and on the upstream side of the valve will be approximately elevation 1435, at which time the valve will be fully open. The

pressure for operating the valve control will be obtained from the upstream side of the valve beyond the taper reducer at the valve. Figure 3 shows the cone valve installation at Station 298+00.

The operation and control of the cone check valves in the well houses, which is an integral part of the well pump operation control, is as follows:

(1) Starting of the pump is accomplished by direct starting of the pump motor through a magnetic type starter, either by remote supervisory control or by an auxiliary start button at the starter.

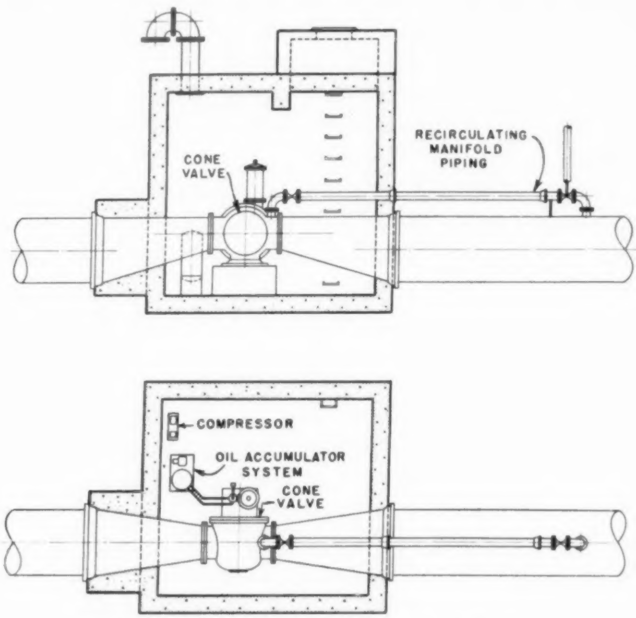

Fig. 3. Automatic Cone Valve Installation

Because the cone check valve is closed from the previous operating cycle, pressure is built up on the pump side of the valve. This pressure operates a contactor having maintaining contacts which close a circuit actuating the cone valve control and the valve is caused to open by means of pressure from an oil accumulator system.

(2) Stopping of the pump is accomplished by first closing the cone valve either by the supervisory control or by means of an auxiliary manual stop button, which, in either case, opens the maintaining contacts on the contactor, de-energizes the cone valve electrical

control and causes the valve to close. When the valve has closed about 90 per cent of its travel, a tripping switch actuated by the valve movement is opened, causing the motor-starter contactor to open and the pump motor to stop. Pushing the emergency stop button or any other interruption of the pilot circuit such as oil pressure failure, overload or power failure will stop the pump and close the valve simultaneously.

Each of the automatic cone check valves in the pump houses and the cone regulating valve in the control station at Station 298+00 is to be operated by means of an oil type accumulator system consisting of an oil accumulator tank, oil pump, air compressor, and automatic oil and air control devices. The control valve at the filtration plant will be hydraulically operated by means of water pressure obtained from the city distribution system.

The rate of opening of the cone check valves is adjustable from two to ten minutes, and the rate of closure is adjustable from two seconds to two minutes.

The five isolating valves installed in the main supply line are manually operated. Cone type valves were selected for this service instead of gate valves because: (a) manual operation of valves 36 in. and larger in size is slow and difficult and no power was available for either hydraulic or electric motor operation; (b) reduced sizes of cone valves installed above long taper fittings can be used with no increase in head loss over that caused by full size gate valves; (c) smaller vaults were required to house the cone valves. All cone valves and angle needle type pressure relief valves are being furnished by the S. Morgan Smith Company.

Supervisory Control of Well Pumps

In order to secure the most flexible and economical operation of the water supply wells, all normal control of the well pumps is centered at the filtration plant in Wichita. Manual control was selected in place of automatic control because it is more flexible, provides more economical pump operation by maintenance of operating heads at a minimum and permits special operation of the pumps during emergencies and unusual conditions. The equipment selected was a 2-wire supervisory control with dispatcher's station and substation equipment at a control station in the well field. The equipment is similar to supervisory equipment used by large electric systems to control distant unattended substations.

To assist the operators at the filter plant there is included an indicating flow meter showing rate of flow into the plant, a recording depth gage for the clear well, and a 2-pen recording pressure gage showing pipe line pressure at the entrance to the plant and also at the lower end of the well field. A dispatching local battery type telephone system connects the filter plant, field control station, the field operator's residence, and all well houses.

The supervisory control equipment is General Electric Polaricode, operating on 125 volt d.c. from storage batteries at the filter plant and at the field control station. The control equipment is suitable for the control of a maximum of 30 wells.

All control lines consist of No. 19 gage paper-insulated copper conductors of conventional lead covered aerial telephone type. The cable is being installed on existing poles of the local telephone company in Wichita, and on separate city-owned pole lines from the city to the well field. In the well field the control cable is being installed on the power lines to the well pumps.

Overhead cable construction was used in preference to the cheaper open type because of the possibility of inductive interference from power lines (6,900/12,000 volt), increased reliability and to permit future joint construction with other utilities. The attenuation of this type of cable required the use of loading coils in the telephone circuit.

One separate pair of conductors extends the entire length of the line from the filter plant to the most distant well with all telephones connected to this circuit. Another pair of conductors is used for telemetering service, transmitting the pipe-line pressure from the lower end of the well field to the recorder at the filter plant, a distance of approximately 22 mi. and to an indicator at the field control station located approximately 7½ mi. in the opposite direction. At least one spare pair of conductors is provided at all points in the cable for use with future wells and for emergency use in case of damage to the active conductors.

The supervisory control equipment operates over two wires between the filter plant and the field control station near well No. 14, a distance of approximately 30 mi. by the cable route. Over these two wires is sent a continuous indication of the condition of each well pump (running, stopped, or tripped), and control (starting or stopping) for each individual well pump. Indication is by means of a set of red, green, and white switchboard indicating lamps for each

well and shows only the position of the motor starters. In case several pumps trip off at the same time, the signals are held and transmitted individually.

At the field control station the supervisory equipment is similar to that at the filter plant except that the control of the individual wells is by direct metallic circuits from this point to all wells, the nearest being about 200 ft. and the farthest about 8 mi. Each well circuit has an additional switch which can transfer control of that well from the filter plant to the field station control. This arrangement permits the field operator, who is normally on duty eight hours per day, to lock-out certain wells for repairs or seasonal inactivity so that no accidental operation can be caused by the filter plant operators.

At each well house there is a small switch which transfers the control of that pump from supervisory, either filter plant or field-control house, to local push-button control. This is to protect the field operator from accidental starting by remote control while working on the pumps. A normal local start-stop push-button station operates the pump in conjunction with the cone check valve in starting and stopping, and an emergency stop button is also provided to stop the motor instantly without waiting for the cone valve to operate.

The filter plant control consists of a dispatcher's bench-board with 25 small sub-panels each controlling a pump, and three meters on the back of the instrument panel. Each small panel contains a red "running" lamp, a green "stopped" lamp, a white "disagreement" lamp, an amber "select" lamp, a "disagreement" key with "run-stop," and a "select" key. In addition to the individual controls there is a group of lamps and keys for operating all pumps, including a start lamp, pilot lamp and key, master start and stop keys, bell key, emergency reset key and alarm bell.

Figure 1 shows the location of the supervisory control and power lines. Figure 4 shows details of the power and supervisory lines. Figure 5 is a typical well house layout showing piping, cone valve, and auxiliary equipment.

To present a better understanding of the supervisory control, the following operation is illustrated:

To stop a well pump, the filter operator ascertains that the start lamp is on, the pilot lamp off, and the equipment is in operating order. He then pulls out the select key on the desired pump panel, which causes the filter plant control to send a series of electrical impulses,

similar to those of a dial type telephone, to the field control board. These impulses cause selector switches at the field control house to make contact with the wires to the well house. The coded impulses are repeated back to the filter plant as a check on the operation, causing the amber select lamp to light and inform the operator that the proper selection has been made at the field control station. The operator then turns the disagreement key on the selected pump panel to "stop," lighting the white disagreement lamp on that panel. He then presses the master stop key which sends a "stop" impulse to the field control station, through the selectors and out over the copper wires to the selected pump. Small relays in this well house then operate to open the circuit to the cone valve equipment which slowly

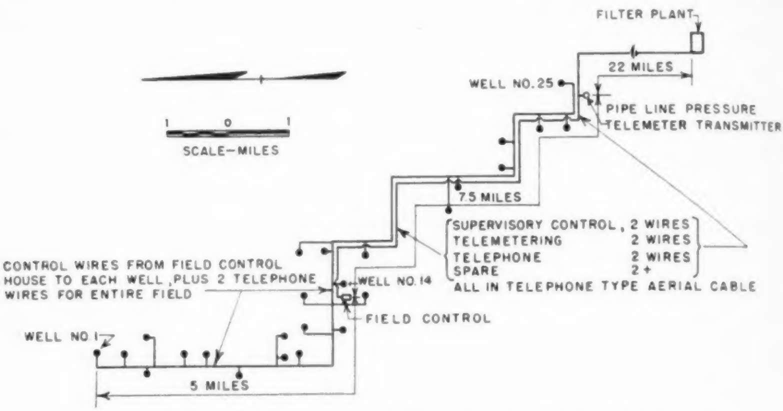

FIG. 4. Detail of Power and Supervisory Control Lines

closes and finally stops the pump motor. When the motor starter opens, a small auxiliary contact opens and then closes, changing the polarity on the indicating wire from this pump. This indication is returned to the field control station where relays send a code impulse back to the filter plant showing that the pump has stopped by lighting the green lamp and disconnecting the red and white lamps on the pump panel. The operator then returns the select key and the control is ready for another operation.

All of these operations require only a few seconds in addition to the time required for the operation of the cone valve, being similar to the functioning of automatic telephone equipment.

Power for the operation of the well pumps will be purchased from the Kansas Gas and Electric Company and distributed by city-owned transmission lines using three No. 4 hard-drawn copper lines. The

total length of the lines will be approximately 16 mi. including the taps into the various well houses. Each well pump will be served from one individual bank of transformers reducing the voltage from 12,000 to 480 volts.

The filter plant being constructed in Wichita is required for the removal of an average of approximately 1 p.p.m. of iron which is present in the well supply. The treatment will consist of aeration over coke tray aerators, lime treatment, coagulation, filtration, and chlorination. The plant is designed for a nominal capacity of 32 m.g.d. using a maximum filtration rate of 4 g.p.m. per sq. ft. of

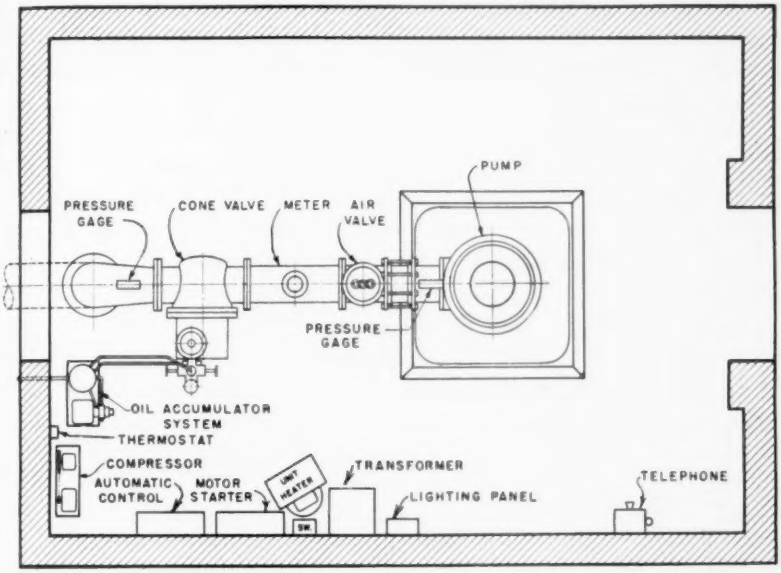

FIG. 5. Well House Installation

sand area. Filtered water storage of 4 mil. gal. is provided at the filtration plant which is connected with an existing 3.4 mil. gal. reservoir of the Wichita Water Company, who will distribute the new supply.

The project is being constructed at a total cost of $2,425,000 of which an amount of $1,091,250 is made available through a Public Works Administration grant. The project is being carried out under the supervision of Mr. Alfred MacDonald, City Manager; Mr. P. F. Brockway, City Engineer; and Mr. M. E. Rogers, Coordinator, for the City of Wichita. Black and Veatch of Kansas City are the designing and supervising engineers.

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2. Wiggin, Thomas H., Enger, M. L., and Schlick, W. J. A Proposed New Method for Determining Barrel Thickness of Cast Iron Pipe. Jour. A. W. W. A. **31**: 841 (May, 1939).
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Discussion by *W. D. Moore*.* The order of "Mono-Cast" pipe for Wichita is the largest order of 48-inch pipe, centrifugally cast, ever manufactured. Some facts as to its production are therefore believed to be of interest and are offered as a supplement to Mr. Lawrence's interesting paper.

The Wichita order of 42- and 48-inch pipe called for approximately 128,000 ft., requiring the manufacture of approximately 2,000 lengths of 42-inch pipe and approximately 6,000 16-foot lengths of 48-inch "Mono-Cast" pipe. The pipe was "tailor made," so to speak, to serve the particular requirements of the several sections of the line, from the standpoint of pressure and earth loading, for safety and maximum economy in the pipe wall thicknesses.

Precision casting and careful inspection were necessary to fulfill the requirements of a rigid specification. The routine control schedule of testing conducted to qualify the pipe was as follows:

First ladle of iron of each heat.	Analysis for carbon and silicon. Chill test. 2-inch round test bar.
Every half-hour during the heat.	Chill test.
Every two hours during the heat.	Analysis for carbon and silicon. 2-inch round test bar.
Last ladle of iron of each heat.	Chill test. 2-inch round test bar. Analysis for carbon and silicon.
Once each heat	Analysis for sulfur, manganese and phosphorus.

* President, American Cast Iron Pipe Co., Birmingham, Ala.

The temperature of the iron as melted was taken once each hour. The temperature of iron was taken for each pipe as poured.

The specifications provided for a maximum working pressure of 100 lb. in the line. A hydrostatic proof test of 250 lb. per sq. in. was applied to each pipe for at least one-half minute, and following this the pipe was subjected to a shock test of 100 lb.

Socket and spigot diameters were checked with circular gages, and the metal thickness of each pipe was checked by calipering.

Other tests were:

Full Length Bursting Test:	From each 300 lengths of pipe as cast, one pipe was tested hydrostatically to destruction.
Ring Crushing Test:	A ring from one pipe for each 50 lengths of pipe as cast.
Talbot Strip Test:	A Talbot strip from one pipe for each 50 lengths of pipe as cast.

The full length bursting test is used to determine the ultimate tensile strength, and is one of the two physical strength factors used in the design of pipe wall thickness.

The ring crushing, or three-edge bearing, test which is made on a ring cut from the spigot end of the pipe, is used to determine the modulus of rupture of the iron in the pipe and is the other physical strength factor used in the design of pipe wall thickness.

The Talbot strip test is a beam test made on a ½-inch wide specimen cut from the walls of the pipe. This test gives the modulus of rupture and the modulus of elasticity of the iron in the pipe and is a measure of its strength and stiffness.

The actual number of tests made during the period of manufacture of approximates 6,000 pieces of 48-inch pipe is given in Table 1.

The total weight of the 48-inch pipe for the entire order was held within a fraction of one per cent of the specification weight. Table 2 gives production performance on this pipe for the period from October 4, 1939 to March 22, 1940.

The thickness of pipe was determined by the use of the new method of design as set forth in the *Manual for the Computation of Strength and Thicknesses of Cast Iron Pipe, A21.1-1939* as approved by American Standards Association, Committee A-21. This method of determining pipe wall thickness takes into consideration such

factors as the physical properties of the iron, working pressure, water hammer, laying conditions, depth of cover, truck load, super-load, impact, etc.

Full advantage may be taken of the actual strength properties of the iron, as determined by tests of the pipe as cast. The Wichita specification provided a tabulation of wall thickness for five different physical strength combinations ranging from 18,000 p.s.i. tensile

TABLE 1

Tests Made on 48-inch Pipe During Manufacture

TYPE	NO. OF TESTS	RESULTS
Chemical analyses of iron as cast	2,610	—
Chill tests of iron as cast	4,161	—
2-inch test bars of iron as cast	1,717	—
Bursting full length pipe	61	23,200 p.s.i. ave. tensile strength
Rings cut from pipe	164	50,800 p.s.i. ave. modulus of rupture
Talbot strips cut from pipe	286	53,500 p.s.i. ave. modulus of rupture 9,400,000 p.s.i. ave. modulus of elasticity

TABLE 2

Production Performance

October 4, 1939 to March 22, 1940

No. of shifts worked (8 hr. per shift)	208
Maximum production for one shift (*lengths*)	42
Total number of lengths cast	5,962
Total number of lengths good	5,861
Total number of lengths rejected	101
Per cent of lengths good	98.31
Per cent of lengths rejected	1.69

and 40,000 p.s.i. modulus of rupture to 25,000 p.s.i. tensile and 50,000 p.s.i. modulus of rupture, using an overall factor of safety of $2\frac{1}{2}$ in each case. As noted in Table 1, the strength properties of the pipe furnished on this contract averaged above 23,000 p.s.i. full length bursting tensile strength and 50,000 p.s.i. ring modulus of rupture. The strip modulus of elasticity averaged less than 10,000,-000 p.s.i.

New Pumping Station at Kansas City

By R. L. Baldwin and C. S. Timanus

A DESIRE to use the most modern and up-to-date equipment to meet the unusual operating problems that exist in Kansas City, Kan., has resulted in a design for the new pumping station, in which many unique features are incorporated. The station has an ultimate capacity of 75 million gallons per day and is the high lift unit that supplies the distribution system.

Perhaps the most outstanding feature of the design is the use of the hydraulic coupling to vary the speed of a synchronous motor-driven centrifugal pump. Two cone-type valves are provided in the discharge of each pump to protect the pumps against severe water hammer in the event of emergency shut-down. The motors are totally enclosed and provided with air coolers; the switchgear is all metal-clad and equipped with air circuit breakers to eliminate fire hazard; and the house transformers are supplied with a synthetic fire-proof fluid instead of oil. Many other features designed for ease of operation and reliability are also included.

The water supply of Kansas City, Kan., is taken from the Missouri River at a point north of the city known as Quindaro Bend. A low lift station, consisting of three motor-driven centrifugal pumps of 30 m.g.d. capacity each and one 50 m.g.d. steam-driven pump, supplies circulating water to the municipal electric generating station located nearby. Passing through the condensers, part of the water is diverted to the water purification plant and the surplus is returned to the river. Following treatment by sedimentation and filtration the

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A paper presented on April 23, 1940, at the Kansas City Convention by R. L. Baldwin and C. S. Timanus, Burns & McDonnell Engineering Company, Kansas City, Mo.

supply passes to three inter-connected clear well basins which provide suction for the high service pumps. This equipment consists of:

(1) One 25 m.g.d. 2-stage centrifugal pump driven through a hydraulic coupling by a 2,000-h.p. 2,300-volt synchronous motor; head, 325 ft.; installed, 1940.

(2) One 25 m.g.d. 2-stage centrifugal pump driven by a 2,000-h.p. 2,300-volt synchronous motor; head, 324 ft.; installed, 1922.

(3) Two 12 m.g.d. crank- and flywheel-type steam pumps; installed, 1910 and 1913.

(4) One 4 m.g.d. centrifugal pump consisting of two separate pumps connected for series operation; each pump is driven by a 150-h.p. induction motor and is designed for operation against a 150-foot head; installed, 1925.

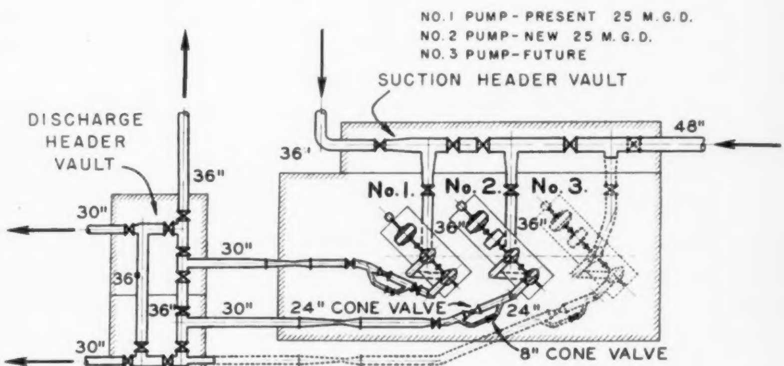

FIG. 1. Plan of the New High Service Pumping Station, Kansas City, Kan.

The total capacity of the present plant is 53 m.g.d., including the two steam pumps and the two small centrifugal units, all four of which are regarded as standby or emergency units.

Station Building

The new station building (see Fig. 1) is planned to house the two large centrifugal pumps and a third future unit of equal capacity. The new 25-m.g.d. pump (see Fig. 2) is now installed and the present 25 m.g.d. unit will be moved into the new station this year.

The pump station building is approximately 110 ft. long by 48 ft. wide. The height above ground is about 35 ft. The operating floor level is 1 ft. above finished grade. The required depth of ex-

cavation was 17 ft., which allowed for a foundation mat 4 ft. thick, making the distance from the basement floor to the operating floor 14 ft.

The building is of reinforced concrete and brick construction with a structural steel framework of the rigid bent type.

Because of its location adjacent to the present pumping station and filter plant it was necessary to remove existing antiquated buildings and to remove and re-locate spur tracks, pipe lines, etc.

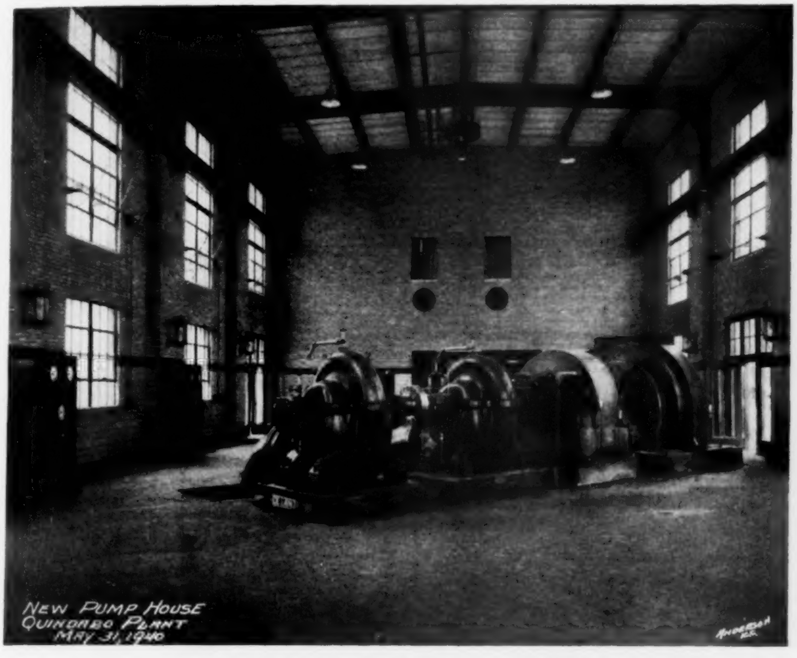

Fig. 2. New 25-m.g.d. Pump as Installed

before excavation could be started. It was necessary to drive steel sheet piling along the entire length of the west wall of the suction header vault to keep an existing spur track in service, as well as to protect a high service pump-suction line. Along the east or front side of the building, existing circulating water piping serving the condensers in the power plant, overhead power lines, etc. were required to be kept in operation throughout the construction period.

The substructure was designed to resist full hydrostatic pressure to a point 2 ft. above the operating floor level, or 5 ft. above the

level of the 1903 flood. The weight of the structure is sufficient to resist flotation to this elevation, and the 4 ft. thick foundation mat is adequate as a beam to transmit the hydrostatic uplift to the sidewalls. The concrete sidewalls below grade are, in general, 24 in. thick. The foundation mat is founded on concrete piles 18 ft. long, driven into the underlying river deposit of fine sand and silt. Piles were jetted in place and were designed for a loading of approximately 25 tons. The piles project 6 in. into the mat and were assumed to be ineffective in resisting the hydrostatic uplift.

The pump foundations are constructed directly on the mat and are completely isolated from the operating floor by a continuous joint. The steel work is of the rigid frame design. As compared to the conventional truss and column design the rigid frame provides the necessary stiffness to resist wind loads, and in addition the depth of the roof beam required was found to be about one-half that of the usual truss construction. These points, together with the fact that the rigid frame is of pleasing lines, uniformly strong, and with no weak details, make it an ideal type of construction for a structure of this nature.

The superstructure walls are of brick construction with exterior face brick and terra cotta trim, and smooth buff-colored interior face brick above a terra cotta wainscot.

The operating floor of the pump room is surfaced with a field of 6 in. x 6 in. red quarry tile laid diagonally, that is, in line with the pumping units. A border of brownish-colored tile is laid parallel to terra cotta wainscoted walls.

In order to provide light in the switch room without the danger of moisture entering through windows, it was decided to use glass block construction.

Another novel feature is found in the design of the roof slabs over the suction header and discharge header vaults. The entire roof slabs of these vaults are made up of removable sections about 3 ft. wide by 13 ft. long and which may be removed when necessary to repair piping or valves. All joints between slabs are sealed and are covered with about 18 in. of earth, which depth is considered sufficient to support a good growth of sod.

Distribution System Hydraulics

As is the case in many other installations, the hydraulic characteristics of the distribution system provided the most difficult

problem in the design of the new pumping station. A diagram (see Fig. 3) indicates the general relation of the pumping station to areas which it serves. Along the low ground adjacent to the Missouri and the Kansas rivers lies the industrial area including the railroad yards and shops, the packing houses, stock-yards, refineries, flour mills, and the wholesale and manufacturing district. Directly south of the station on relatively high ground, along Minnesota Avenue, is the business district or high value area. To the west, southwest and extreme south are the residential areas.

Extending south from the pumping station are three large trunk

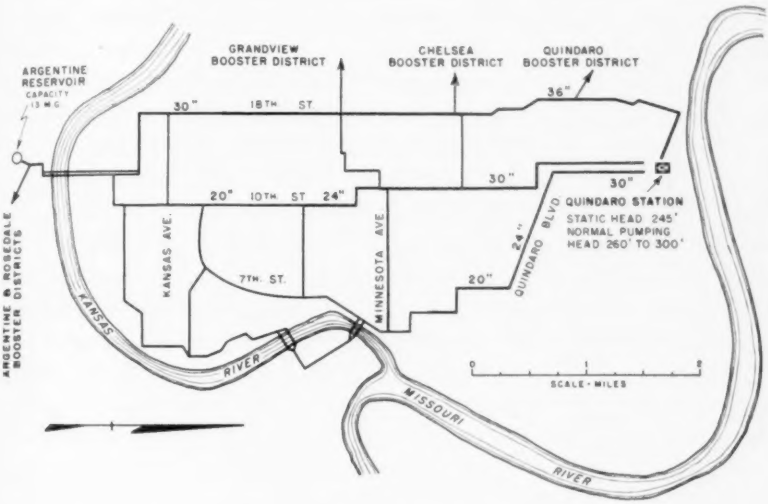

FIG. 3. Diagram of Area Served by New Pumping Station

mains which supply the grid system and a large reservoir of 13 million gallons capacity located in Argentine. To the west and south of the principal grid system are five booster districts which are within themselves complete systems with centrifugal pumps taking suction from the main feeders and discharging to the individual booster districts, all of which are provided with elevated storage.

Ground elevations and operating conditions are such, in the principal grid system, that Argentine reservoir will not in itself provide proper pressure in the business district or high value area. Hence it has become the practice to maintain pressure in this area by par-

tially throttling the flow of water to the south part of the city in the daytime and raising the pump station pressure during the night. This, in effect, diminishes the area supplied by the reservoir and converts the entire north end of the system into a direct pressure district which receives no benefit from the reservoir storage. At night the path to the Argentine reservoir is fully opened and the reservoir supply replenished. During severe periods of draft this is not completely accomplished until Sunday when the low load on the system permits enough water to flow to the reservoir to fill it to the normal operating level.

It is to be observed that this method of operation not only puts a wide variation in demand on the pump station during the day but also imposes an entirely different set of conditions at night. It was for these reasons that considerable attention was given to the type of drive for the new pump. A steam turbine-driven unit was considered but was rejected because the expenditure required for such an installation was approximately $100,000 more than that required for the all-electric station. Then, too, the electric units were considered as reliable as the steam unit because of the reliability and high order of the electric supply, together with the fact that the same electrical supply serves all the motor-driven auxiliaries of the boiler plant which would have supplied steam to a steam pumping unit. To meet the operating requirements that demanded variable speed control on the pump, a synchronous motor driving the pump through a hydraulic coupling was selected. This arrangement offers not only great operating flexibility but also provides power factor correction of considerable value to the electrical system.

The principal consideration in selecting the size of pump was the necessity of firming the existing centrifugal pump.

The crank and flywheel pumps are about 30 years old and occupy a large amount of space that might be useful for other purposes. Also these pumps require the maintenance of an old boiler plant, which has become obsolete, or the maintenance of a long steam line from the new boiler plant from which steam must be reduced in pressure and put through a de-superheater in order to be used.

The location of the pump house was given much study and various layouts were made. Because two sets of pipes (those from the old Kansas City, Mo., pumping plant, now the property of Kansas City, Kan., and those from the existing Kansas City, Kan., pumping units) were already in the ground and because those now in service had to

be kept in operation, the location of a tract large enough to install a new pumping station was fraught with many difficulties.

Station Design and Equipment

The final location selected involved the construction of discharge and suction lines under difficult conditions, under tracks and near other lines under pressure. The suction lines were finally constructed along direct lines and comprise a 48-inch and 36-inch loop-type suction header with valves provided so that repairing or cutting into a section for extensions will cause a minimum amount of trouble.

The discharge connections are made through cone valves, and Venturi meters on each pump discharge into a 36-inch flanged ring header from which three (24-, 30- and 36-inch) main discharge lines are taken to the city. Later a fourth line may be connected. This header is valved so that any pump or any discharge line may be out of service without causing any serious difficulty in furnishing service.

All suction lines are of Class C cast iron pipe and all discharge connections and lines of Class D pipe. The discharge lines from the pumps to points where the three mains leave the discharge header vault are of flanged construction. Bell and spigot joints with lug bolt connections are introduced at various points to provide flexibility and to facilitate removal of valves and fittings. Any valve or fitting may be removed without disrupting service.

All large valves are geared and valve boxes set so a portable motor-driven valve operator may be used to open and close them.

The motors are designed to make it possible to get a maximum of power factor correction. The new motor was specified to be similar to the existing one in general characteristics and rating. This required a 2,000-h.p. motor with 60 per cent leading power factor at rated horsepower, 2,400 volt, 60 cycle, 3 phase, 720 r.p.m. It has sufficient starting torque to bring the unit up to speed with the pump-suction valves open, with the discharge valve closed and the pump full of water and with motor-starting inrush limited to 7,000 k.v.a. when using reduced-voltage starting. The pull in torque is sufficient to pull the unit into step with the discharge valve fully open and with full voltage on the unit.

A direct-connected exciter of 16 k.w. capacity is provided and yields full excitation for the motor when operating at rated horsepower and at 60 per cent leading power factor.

Roughly, the additional cost of the motor and switching equipment due to power factor correction was $5,000 and the possible added generator capacity in the adjacent power plant, due to its use, will be approximately 1,200 k.w. which is worth at least $25,000.

It was also desired to eliminate as much of the roar, usually accompanying the operation of large synchronous motors, as possible. The motor was provided with a closed cooling system, all ventilating air being recirculated through an air cooler located below the motor in the same manner as such coolers are installed beneath modern turbine-driven generator installations. This not only prevents a large part of the noise but also eliminates the ordinary dirt and oil deposits on windings, which deposits occur where the room air is used for cooling purposes.

Cooling water for the air cooler is taken from the first stage of the pump and returned to the suction. The fin-tube surface air cooler has a cooling surface of 2,100 sq. ft. suitable for cooling 11,500 cu. ft. per min. of ventilating air required. The water required for cooling is about 95 g.p.m. Cooler size is ample for cooling even with the 95° to 100° F. water and air temperatures which occur in the vicinity during the summer months.

The pump casing and connections are designed so that, whenever necessary, the impellers may be changed for 30 m.g.d. capacity, and motor capacity is ample for such service.

The motor is provided with shaft length and bed plate designed for a stator shift so that the stator may be shifted over for inspection or rewinding without removal of the rotor.

Hydraulic Coupling

The hydraulic coupling is designed for heavy duty and is provided with an oil pump and motor for forced bearing lubrication, a reversing pump and motor for speed control, and an oil reservoir and oil cooler. The oil cooler is located below the bed plate in the basement and is cooled by water which is taken from the first stage of the pump and then returned to the suction. Speed control is effected by varying the amount of oil in the coupling. This is accomplished by using the small reversible pump which is designed to transfer oil from the reservoir to the coupling, thereby increasing the amount of oil in the coupling and raising the speed or cutting down the slip; or from the coupling to the reservoir, thereby decreas-

ing the amount of oil in the coupling and lowering the speed or in creasing the slip.

The cooler requires about 65 g.p.m. of water. The water pressure in the first stage is normally more than 50 lb. per sq. in. while oil in the cooler is at very low pressure, i.e. only that required for circulation; so that ordinarily the water pressure will be such that, should leakage develop, it would be from the water into the oil rather than the reverse. To prevent any possibility of oil or leakage reaching the suction, an automatic three-way cock is installed in the drain. It is actuated by the water pressure maintained on the oil cooler. This is set so that whenever pressure drops below 20 lb. the valve diverts flow to an open waste and it can only go to the suction when pressure is 20 lb. or more on the cooler.

The hydraulic coupling was introduced in England in 1930 and brought to this country two years later. It has been widely used for forced and induced fan drives in boiler plants but is new to the water works field.

There are two radially-vaned members that make up the main rotating parts of the hydraulic coupling. The primary member or impeller is mounted on the driving shaft, and the secondary member or runner is mounted on the driven shaft. These members actually are separated by about $\frac{1}{4}$ in., they have straight radial blades and cast integrally with these blades there is a core or guide ring of semicircular section with one-half in the impeller and one-half in the runner. The oil being forced by the impeller, through the runner, causes it to rotate or drags it around in the same direction it rotates. If the space is filled with oil the slip is only 2 or 3 per cent but if the amount of oil is reduced the slippage increases until with no oil at all, and only air to act as the dragging medium, the speed will drop to about 20 per cent.

Inner and outer casings are carried by the impeller and the oil naturally takes its place in an annular ring in these casings, that is the portion that is not being driven through the runner.

Calibrated ports are provided at the periphery of the inner casing and a certain amount of oil flows continuously through these ports into the space between the outer and inner casing where it forms an annular ring. This rotating ring of oil impinges upon the opening of a stationary scoop tube which forces it through passages and pipes to the oil cooler, and thence back into the working circuit again. This means a continuous change of oil, withdrawing oil

heated by friction and adding cooled oil, so making continuous operation possible.

Automatic Starting Equipment

The starting of a large synchronous motor requires careful synchronizing in order to eliminate severe disturbances on any electrical system. For this reason starting is done automatically. Each pump control circuit is complete so that turning the motor starting switch to the starting position starts the motor and carries the starting sequence to completion, including opening of the pump discharge valves. The starting equipment is of the reactor type. All high voltage switches are of the new air-breaker type eliminating the usual fire hazard attached to the ordinary oil-insulated circuit breaker. The switchgear is of the type known as "metal clad." The circuit breakers are of the horizontal draw-out type. Each motor is provided with a starting unit, which connects the motor through the starting reactor to the main bus, and a running unit which connects the motor directly to the main bus. Compartments are provided for instrument or relay transformers, for disconnecting devices completely separating all high and low voltage circuits.

The switchgear consists of the following units: station service circuit; starting and running positions for motor #1; incoming line #1; bus tie; incoming line #2; and starting and running position for motor #2. Blank panels are also provided for future additions which will include: bus ties; incoming line #3; and starting and running positions for motor #3.

Bus compartments are of $\frac{1}{8}$-inch steel sheets, each bus being separated from the other by steel barriers. Instruments on the main switchboard are flush mounted and all instrument wiring and instruments or relays may be easily inspected or tested. Circuit breakers are withdrawn from the back of the structure.

The south end of the pump house is provided with a room to house the switch gear. The switch gear is set so that its front is approximately at the wall surface and the side of the room next to the pump room is open and framed to fit the switch gear dimension, leaving about $\frac{3}{4}$ in. clearance. This gives the appearance that switchboard is mounted flush with the wall. The starting reactors are located along the back wall of the switch room and there is ample space between the switch compartment and the reactors for withdrawal of the circuit breakers for inspection and maintenance purposes.

On the second floor at the south end of the building are located a transformer room for furnishing house service, a battery room for station service, and a room for office or storage.

The station transformers reduce the voltage from 2,400 volts to 120/240 for lighting and 480 for small power. These transformers are insulated with a synthetic fluid which is fireproof and non-explosive so that it is unnecessary to use fire doors and special cell construction for their installation inside the building.

The battery is 60 cells with an 80-ampere hour rating and is equipped with a copper oxide rectifier-charger for floating operation and for recharging. The battery serves control circuits and special emergency lights.

The 10-ton 3-motor crane is operated by 480-volt power. The same power is used for window sash operation, two vacuum pumps, hydraulic coupling motor, suction and discharge valves and sump pump.

Duplicate motor-driven priming pumps are provided, each having a capacity of 125 cu. ft. per min.

The power supply for the station is taken through underground cables connecting the new pumping station to both of the adjacent power plants. A 2,400-volt 3-phase power circuit comes from each plant, and conduit space is available for a third circuit.

A 480-volt power circuit is also carried underground from the power plant to the pumping station to duplicate the supply from the station transformers.

Conduit runs were installed under difficult conditions across main spur tracks supplying coal to the power plant and under 60-inch circulating water lines. The manhole construction made it necessary to take one track entirely out of use for several weeks. Four and one-half inch fiber ducts are laid in concrete and they are planned to care for future extension both to this high service pump station and to the low service station.

Connections and switching facilities are provided so that either 2,400-volt circuit can supply energy to either pump separately, if desired or necessary. The supply will be to a bus from which pump motors receive their energy. If the bus-tie circuit breaker is open, however, the pump motors are on independent circuits in the normal manner of operation.

Opposite each pump at the wall is a small control panel from which the entire operation originates. This panel has two indicating

gages (one showing suction pressure, the other discharge pressure, at the pump nozzle) and a flow meter indicator, recorder and totalizer. On it is also located the pump-motor starting and stopping switch, indicating lights for the main cone and small cone valves, priming-pump control, and gate valves on both suction and discharge lines.

Starting and Stopping Operations

Motor starting, as mentioned before, is automatic. The excitation is applied at the most desirable position of rotor and stator to give powerful synchronizing with minimum current disturbance. Use is made of electronic tubes to select this point, the method being known as "angle switching."

The starting operation is as follows:

(1) As soon as pump has been primed, press "Start" button. This synchronizes the motor and a contact on the field contactor energizes the solenoid pilot valve on the 8-inch by-pass cone valve. Another set of contacts on the motor field contactor controls the oil pump on the hydraulic coupling, during the starting sequence, until the pump has been speeded up sufficiently to close the pressure switch set at some pre-determined value above static pressure.

(2) When the by-pass valve reaches its limit of travel, a limit switch energizes a three-way solenoid valve in the "opening pitometer" line to the main valve, thereby permitting the latter to open. The main valve is now in position to operate as an automatic check valve. The rate of pumping—that is, the amount of oil in the coupling—is now controlled by a push-button station located on the pump control panel. An excess-pressure alarm will ring when a pre-determined pressure is reached, thereby notifying the operator that the pump speed is to be reduced.

In stopping, the rate of discharge is reduced very gradually to about 10 m.g.d. (by means of hydraulic coupling on the new pump, or with a throttling valve on the old pump). Pressing the "Stop" button will then set up the following sequence:

(1) The main valve closes as a check valve as soon as the three-way solenoid valve in the "opening pitometer" line of the main valve is de-energized by pushing the "Stop" button.

(2) The by-pass valve closes at a rate slow enough to prevent any appreciable pressure rise. Complete rotation of the plug will be accomplished in about 3 minutes. The main valve is inter-

locked so that it will close if the by-pass valve moves from the full open position for any reason.

(3) With the by-pass valve closed and the pump churning, the motor automatically trips out.

In the event of a local or general power failure, or of any mechanical failure which causes the source of power to the pump to be instantaneously interrupted, the following sequence is set up:

(1) The main valve is operating as a check valve and closes due to the drop in pressure on the pump side, or due to fact that the 3-way solenoid valve in the "opening pitometer" line is de-energized.

(2) The by-pass valve slowly closes as in the case of normal shutdown just described. The maximum rate of reverse flow through the pump with the by-pass valve fully open is estimated to be about 12 m.g.d.

There is much information which will be available at a later date regarding the performance of the various items of equipment. A high speed recording pressure gage will be used to study all problems of water hammer and pressure rise due to the closure of valves and the starting and stopping of the units. Detailed studies will also be made on the effect of variation in speed. From this information standard methods of operating procedure will be established to effect the most economical and reliable results.

Leak Detection

By H. W. Niemeyer

LEAK detection is an important adjunct of the water supply business, the extent of that importance being determined by the cost of delivering the finished product and by limitations of the supply. It is a component of the efficient operation of any distribution system and one which has been accentuated by meterization of customer services.

Unfortunately the various factors contributing to the difference between pumpage and metered uses are compounded into a single total figure. The separate losses can never be accurately known and reported so that the plant operator can determine when and where to apply the proper corrective measures. Lack of attention to one phase of distribution system operation will discredit the results of efficient practices applied to other parts. All of the phases must therefore be given attention not only to reduce unmetered water to a minimum but to allow the operator more accurate information particularly regarding meter performance and meter practices.

Leak detection is an art that had its inception with the advent of underground pipe lines. Hard surface roadways (which are becoming more general and better constructed) hold in leaks which then find a path of lesser resistance to an underground crevice or a nearby sewer. With corrosion, settlement, water hammer and construction faults the continuous enemies of pipe lines, leaks are always in a state of development and a large percentage of them will fail to show surface indications. Consequently, most distribution systems are subject to constant loss resulting from an aggregate of many

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A paper presented on April 4, 1940, at the Indiana Section Meeting, Lafayette, Ind., by H. W. Niemeyer, Superintendent of Distribution Department, Indianapolis Water Co., Indianapolis.

small leaks, and possibly some large leaks, unless continuous leak surveying is practiced.

There are two general methods of leak surveying available, the measurement and the sound-pick-up methods. Either can be used effectively and in some cases it may be necessary to use both methods in conjunction with each other for effective results.

Pitometer Surveys

One measurement method of leak detection is practiced by isolating a section of a distribution system and measuring by pitometer the input into it over a period of time while at the same time the metered consumption of customers in the section being surveyed is recorded. It must be pointed out that this method detects illegal use of water and inaccurate meters as well as leakage losses and can be further used to determine the hydraulic characteristics of the distribution system. Pitometer surveys considered with these other purposes in mind are not to be confused with straight leak-detection surveys.

A section of pipe line can be tested quickly for leakage by shutting it out and feeding it through a hydrant with a hose from another hydrant on a live part of the system. A standard meter is inserted in the hose line. If possible, all services connected to the pipe in test are momentarily shut off and movement of the meter then observed. If leakage is detected by either the meter or pitometer method, however, the leak or leaks must be run out to exact location by listening in on various points of the line.

Detection by Sound Pick-Up

The sound-pick-up method of leak detection is, of course, made possible by the fact that fluids escaping under pressure from an orifice will set up vibrations in a pipe line. These vibrations are audible to the human ear depending on certain conditions. The practice of "listening in" has therefore been used with a certain degree of success by merely establishing contact between the ear and exposed parts of the distribution system. Devices giving mechanical amplification of the sound vibrations have been of aid in leak detection, some having sufficient power to allow detection through soil when vibrations are strong. During recent years the principle of electrical amplification has been applied to sound pick up with such effective results that fast, economical, and efficient leak surveying is now possible.

The principle of the radio leak locator is simple. A crystal microphone is placed in contact with a part of the distribution system which converts the pipe vibrations into electrical impulses that are then amplified by radio. The electrical impulses are measured in the final stage and also converted into an audible signal in head phones. It is quite obvious that vibrations far beyond the limits of audibility of the human ear can be detected in this manner. A leak indication is run out to its source by taking comparative readings of the measured vibrations at various points along the pipe line by direct contact either on exposed parts or with probe bars; or by surface readings on the ground over the pipe.

Operation of Radio Leak Locator

The operation of the radio leak locator is not quite as simple as it sounds because of several influencing factors. First, the main pressure, pipe size, and the size, shape and submergence of the leak orifice determine the frequency and amplitude of the vibrations given off. Next, the physical characteristics of the pipe line and soil surrounding it force the vibrations to follow certain laws of physics governing direction and effectiveness of transmission. Pipe size, pipe material, pipe bends, and ground dampening all control the transmission of vibrations. Third, mechanical or electrical vibrations may exist that will be amplified along with those given off by the leak. While this condition is partially corrected by the introduction of electrical filters in the amplification process, vibrations of the same frequency as that given off by the leak cannot be eliminated. Lastly, the locating instrument may develop defects that will affect readings without becoming apparent to the operator. Therefore, the success of leak surveying with this type of equipment depends very much upon judgment and training of the operator in allowing for the variable factors.

It must be emphasized that leak detection is an art. While some may be born artists, generally the leak locating personnel must develop proficiency by actual experience. This is particularly true of the operator of amplifying equipment. If the highest efficiency in detection work is to be attained, then the equipment must be assigned to a regular operator. There is little value in discussing operating details of leak locators. Results of survey operations in Indianapolis, however, will be given to indicate the effectiveness of such equipment.

Survey Method

The general method of survey is as follows: First, a regular man is assigned to operate the detection equipment. This equipment is carried in an especially outfitted delivery sedan, which unit carries a spot light for night work, a gasoline-powered pavement breaker, and miscellaneous street tools necessary to handle services, valves, hydrants and to do pipe probing work. A laborer assists on the work.

The surveys so far have been conducted by making instrument contact with hydrants only, except that valves are used for contact points on feeder mains having no hydrant connections. Hydrants make an excellent detection contact because sound vibrations form a peak at the end of a pipe structure. This is fortunate, since they provide the most convenient connection possible. Leak indications, when obtained, have been traced to location by contacting services and valves or pipe as necessary. After gaining experience, the operator usually knows, from the indication obtained, the approximate location to start working. This method of survey has allowed great efficiency; the entire system (6,200 hydrants) has been covered in a 5-month period with all indications run out to leak location.

Results of Surveys

During the two years in which surveying operations have been carried on, a total of 250 leaks have been uncovered with the radio equipment. Very few of this number gave surface indications. In fact, the largest percentage of waste was found to be getting away in sewers and probably would never have appeared on the surface. Leaks were found on 25 dead-end service lines, many of which were running at capacity. The total waste by all leaks, as estimated at the time of repairs, reached a total in excess of 3 million gallons a day. The size of leaks varied from drips (at hydrant bases) to one of $1\frac{1}{4}$ million gallons per day. This latter leak, which evidently was caused by electrolysis, lay under 11 ft. of cover with the advantage of a 30-inch storm sewer within 2 ft. of the main. The efficiency of the equipment is best indicated by the fact that leak indications were obtained on hydrants 500 to 600 ft. away from a leak of 10 gallons per minute.

The radio leak-surveying system supplements a continuous "listening in" program that is used by the meter readers on the service

lines at the meter. I mention this auxiliary work of the readers for it has proven to be very worth while. Over 500 service line leaks including those at stop and waste valves are reported annually by this group. Since they cover the entire town monthly, waste from customer service lines is held to a minimum.

In addition to use of regular survey methods, all water plant employees should be made to be alert to leak indications. Surface water, damp spots in the ground, excessive amounts of water running in sewers during dry seasons, water standing in valve boxes or pits, are all good indicators and should be watched for by employees for reporting and investigation.

In Indianapolis, as in other cities where leak surveying has been practiced, immediate benefits have accrued by a reduction of water waste. In addition, more positive knowledge of distribution system condition has been gained so that an intelligent allocation of unmetered water can be made. While no plant can ever expect to attain the perfection of a 100 per cent accounting for pumpage, it should be borne in mind that the total efficiency of a distribution system is determined by its approach to this 100 per cent mark.

The World's Largest Filtration Plant

By Loran D. Gayton

CHICAGO now has under construction the largest rapid sand filtration plant designed or built up to the present time. At the usual rating of two gallons per minute per square foot of filter sand area this plant will have a capacity of 320 million gallons per day and it is being designed to handle a peak hour demand of 450 million gallons per day.

During the years from 1933 to 1936, the City presented to the Public Works Administration five applications for either grant and loan, or grant only, in connection with the construction of the South District Filtration Project. These applications were for financial assistance in connection with the construction of the complete filtration project. In July 1938, the Honorable Oscar E. Hewitt, Commissioner of Public Works of the City of Chicago, again applied for a grant in connection with the Filtration Project, and in August, 1938, the P.W.A. approved a 45 per cent grant in connection with $12,035,000 worth of work. This $12,035,000 was the estimated value of the amount of work that it was considered could be carried out within a period specified by the P.W.A. Since approving the original grant, the P.W.A. officials have agreed to certain changes in the various items of work and have also granted the City an extension of time to carry out the construction.

As it now stands, the following units of work are included under the grant: the breakwater, the bulkhead or cofferdam, the park fill, the approach fill, the filtration plant tunnels, the east substructure, the west substructure, the low lift pumps and motors, certain cast iron pipe and fittings, certain sluice gates and valves, Venturi tubes and recorders, and the erection of these items. Under the present

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A paper presented on May 23, 1940, at the Illinois Section Meeting, Chicago, by Loran D. Gayton, City Engineer, Chicago.

agreement with the P.W.A. it is required that all the foregoing work be completed by March 1, 1941.

The South District Filtration Project was started in the fall of 1938 and to date over $11,000,000 worth of work has either been completed or put under contract (see Table 1). In addition to the foregoing, bids have been received and contracts are in process of execution for a total of $466,000. The entire South District Filtration Project, including metering the entire area served by the plant, when completed, will represent an investment of over $20,000,000.

Preparation of Plans and Specifications

In preparing the plans and the specifications for that part of the South District Filtration Project coming under the present P.W.A. grant, it is believed that the Bureau of Engineering made an enviable record as regards the shortness of the time required, and the completeness and adequacy of the documents submitted to prospective bidders.

On August 3, 1938, when the P.W.A. grant was accepted by the Chicago City Council, there was available the data necessary to determine the general dimensions of the proposed plant and its required capacity, but on this date not one of the contract drawings had been started.

On August 10, 1938, the City Engineer was authorized to prepare plans and specifications for a rubble-mound type of breakwater, which was the first unit of the project required. These plans and specifications were prepared by the regular organization of the Designing Division of the Bureau of Engineering, and submitted to the Commissioner of Public Works on August 13, 1938, or three days after the necessary authority had been given. These drawings were submitted to the P.W.A. officials in Chicago on August 15, 1938. The contract was advertised on September 7, 1938, bids opened on September 19, 1938, and work started September 21, 1938. The contract amounted to $709,825.

The work of organizing a separate filtration section in the Designing Division of the Bureau of Engineering was initiated during the latter part of August, 1938; this filtration section was organized to prepare all the necessary plans and specifications in connection with the proposed South District Filtration Project. On August 29, 1938, the first three engineers started work, and this force was gradually

built up until by December 1, 1938, there were 37 engineers and 13 clerical personnel employed.

The second unit required for the project was the bulkhead or cofferdam, which was to enclose the site of the filtration plant proper. These plans and specifications were started by the filtration section of the Designing Division in September, 1938, and were transmitted

TABLE 1

South District Filtration Project

*Status of Contracts as of July 1, 1940**

CONTRACT	CONTRACTOR	AMOUNT OF CONTRACT	STATUS
Breakwater	Great Lakes Dredge & Dock Company	$709,825.00	Complete
Bulkhead	Fitz Simons & Connell Dredge & Dock Co.	1,167,250.00	Complete
Park Fill	Fitz Simons & Connell Dredge & Dock Co.	983,100.00	66% complete
Tunnels	Wenzel & Henoch Co.	1,589,200.00	56% complete
Approach Fill	Fitz Simons & Connell Dredge & Dock Co.	116,200.00	Complete
E. Substructure	Michael Pontarelli & Sons	3,616,650.00	62% complete
W. Substructure	Michael Pontarelli & Sons	2,231,500.00	10% complete
Low Lift Pumps	Allis-Chalmers Mfg. Co.	79,700.00	Contract Awarded
Motors	Electric Machinery Mfg. Co.	44,250.00	" "
Sluice Gates	Mueller Company	166,232.12	" "
Cone Valves	Chapman Valve Mfg. Co.	57,420.00	" "
Cast Iron Pipe & Fittings	United States Pipe & Foundry Company	166,101.34	" "
Gate Valves	Chapman Valve Mfg. Co.	286,275.00	" "
Venturi Meters	Builders Iron Foundry	51,000.00	" "
Total		$11,264,703.46	

* Erection Contract—Bids opened July 2, 1940.

to the P.W.A. officials on October 17, 1938. The contract was advertised on October 26, 1938, and bids were opened on November 14,1938. The bulkhead is of an unusual design; it encloses an area of approximately 38 acres, and its perimeter is approximately one mile in length. The contract price was $1,167,250.

From the fall of 1938 to the fall of 1939, the filtration section of the

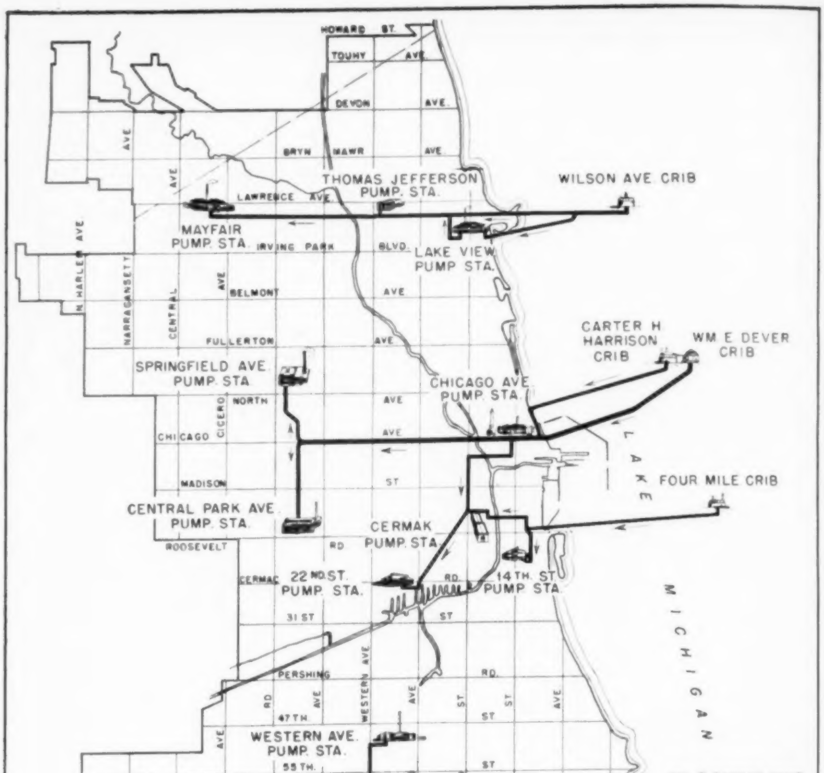

FIG. 1. Chicago's Water Supply System, Showing Location of South District Filtration Plant

Designing Division prepared plans and specifications covering seven separate contracts, these individual contracts ranging from approximately $116,000 to $3,616,000 with a total value for all contracts of almost ten and one-half million dollars.

At the present time, on the design of the project, there are employed 70 engineers and 11 clerks and, supervising the construction, the city's force consists of 81 engineers and 3 clerks. The various contractors are now employing approximately 1,800 men on the project.

Population and Area

The South District Filtration Plant will supply filtered water to three South Side Pumping Stations, the 68th Street, the Roseland and the Western Avenue Stations. The area within the city limits to be served by this plant, covers 115 square miles or 54 per cent of the area of the entire city. The present population in this area is 1,387,000 or 38 per cent of the city's total population. It is estimated that in 1950, the population will be approximately 1,769,000 and in 1960 this figure should approach 2,000,000.

For the past ten years, under a partially metered system, the average daily per capita consumption has been 264 gallons. With 100 per cent metering the average per capita consumption should be reduced probably to below 200 gallons per day. It should be kept in mind, however, that the area under consideration has possibilities for great industrial expansion and the industrial use of water may keep the average per capita consumption somewhat high.

Plant Capacity

The capacity of the plant is based on a maximum average daily filtration rate of 2.5 g.p.m. per sq.ft. in winter and 3.0 in summer. These rates are based upon the use of acid-treated sodium silicate in the treatment of the water to strengthen the coagulation during periods when the floc produced by the use of a simple coagulant only would not be strong. These rates may be exceeded for periods of several hours daily with 3.0 g.p.m. being a maximum winter rate and 4.0, a maximum summer rate. Because of the high friction loss through the sand at or above the 4.0 g.p.m. rate, this rate should not be exceeded nor should an attempt be made to maintain it continuously throughout the day. Should a coagulant only be used, the maximum winter rate should average not more than 2 g.p.m. for the day and should not exceed 2.5 at any time.

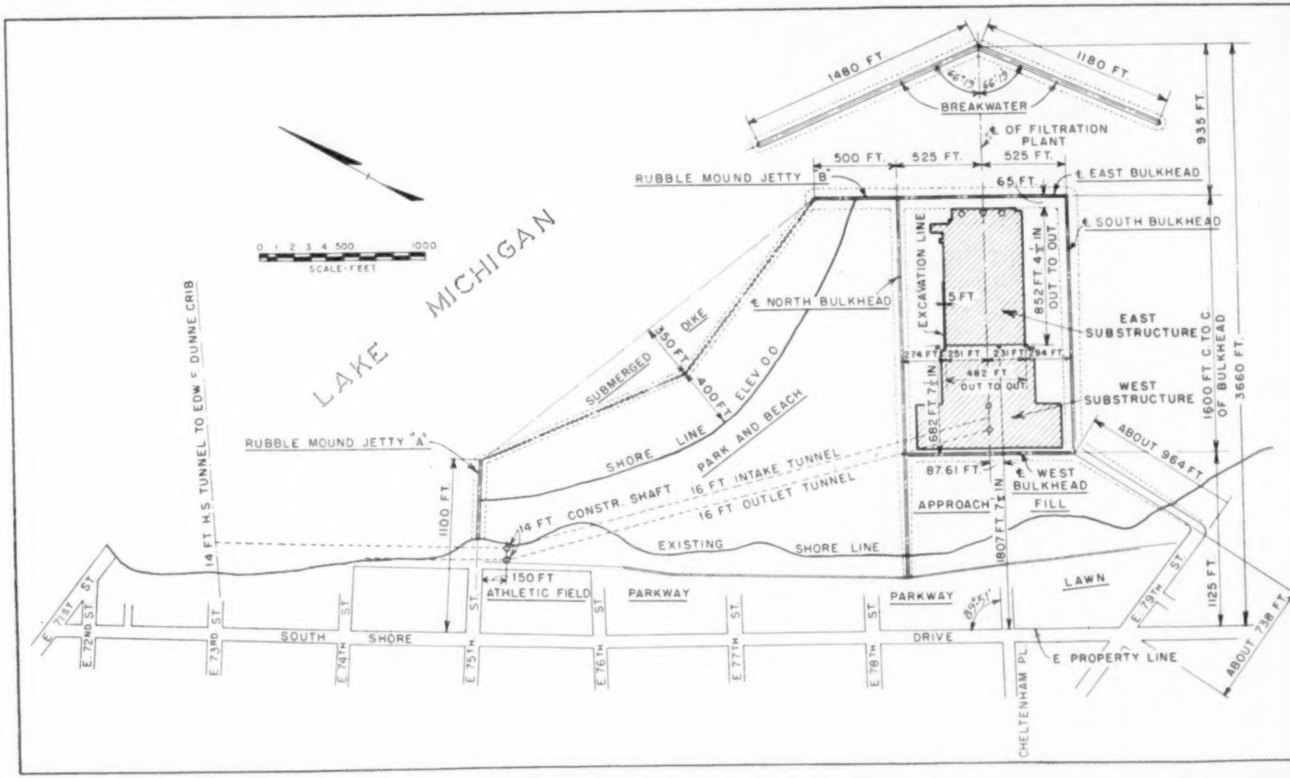

FIG. 2. General Plan of the Project

The time of coagulating after addition of the coagulant to the water will be 58 minutes for a 280 m.g.d. yearly average of water treated. This will give 45 min. mixing time for the average of the maximum summer day of 360 m.g.d. and about 35 min. for the peak hour demand of 460 m.g.d. The time of settling will be 4 hours for the yearly average of water treated, which time will be reduced to $2\frac{1}{2}$ hr. at the peak rate.

Water Flow Through Plant

Raw water from the Edward F. Dunne Crib passing through the filtration plant intake tunnel, enters an intake basin at the east end of the plant, several feet below lake level, by gravity, and is screened before entering a suction well at the low lift pump.

Four 50 m.g.d. and four 100 m.g.d. direct-connected, electric motor-driven, centrifugal pumps are provided to care for a peak load of 450 m.g.d. Various combinations of these pumps will care for fluctuating loads and provide spare units to guarantee uninterrupted plant operations. The flow from each pump is recorded by a Venturi meter.

Each low lift pump raises the raw water from lake level to an elevation 20 ft. above, in a double decked raw water conduit. This double decked design of the raw water conduit permits either or both levels to be used and permits cleaning of the conduit without interruption of plant operation.

From the raw water conduit, the raw water is metered to each of the three mixing, coagulating and settling basins.

Any one of the three units may be taken out of service for cleaning or repairs. Two will provide the average daily maximum requirements.

The settling basins are 33 ft. deep but have an intermediate settling floor at one-half their depths. Over these intermediate floors, as well as over the floors at the bottom of the basins, sludge scrapers are provided to drag the sediment formed to one side of each basin where cross collectors convey the sludge to sumps in which sludge connections provide continuous sludge removal.

This sludge runs by gravity to a sludge well and concentration tank where, after concentration, the water is laundered off and returned to the raw water intake while the sludge itself is removed for further concentration and disposal. Thus the treated water and its chemicals are reclaimed.

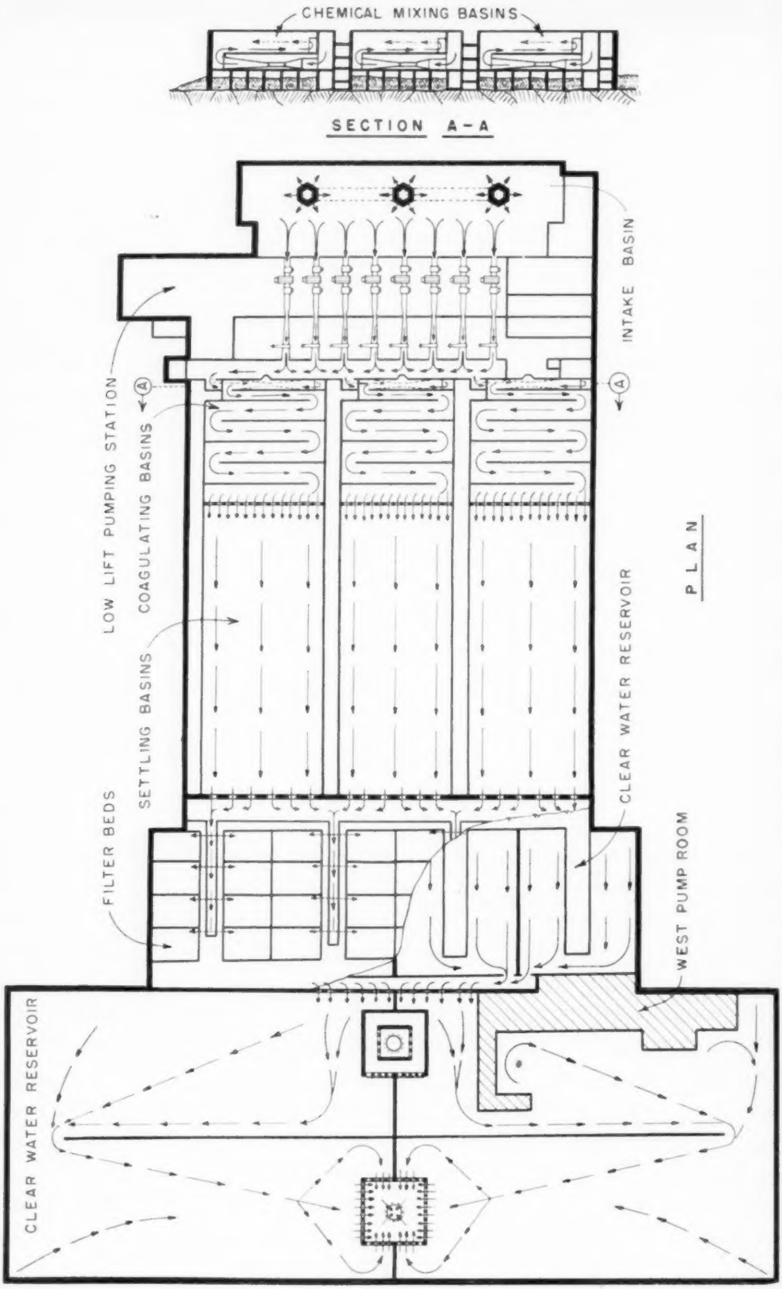

FIG. 3. Plan of the Filtration Plant, with flow of water indicated

All filter wash water is returned by gravity to a sump, from whence it is pumped to the wash water settling basins, where the sludge is settled out and the clear water is decanted over into the low lift pump suction well. The sludge is added to that collected from the settling basin for further concentration and disposal.

At the end of each settling basin a recarbonation chamber is provided where CO_2 may be added. The treated water then passes through sluice gates into a settled water header in the filter building. Four settled water laterals branch off this header and, extending under the filter gallery floors, distribute to four units of 20 filters each. Each unit of 20 filters discharges into a separate clear water reservoir located below the filters. From this point the filtered water passes into a clear water header which discharges into the main clear water reservoir, so arranged and divided by sluice gates that either half or all of the reservoir may be used. From the reservoir the filtered water enters the outlet shaft and then goes to the underground tunnels which supply the pumping stations.

The chemical application and mixing basins are located above the meters, and each basin extends nearly the full width of the coagulating basin. Most of the chemicals will be added to the raw water in these basins. Mixers for producing rapid initial mixing of the chemicals with the water are located therein.

Baffle Chambers

In addition to the rapid initial mixing, the water in each unit of the plant will flow through another chamber extending the height of both the meter and chemical application basins. The depth of water in this chamber will be approximately 35 ft. The chambers will be provided with obstructions to produce additional rapid mixing of the chemicals with the water. The obstructions will be so placed as to produce approximately a uniform flow of water from top to bottom of the chamber. Near the outlet end of this chamber a dividing floor between the upper and lower coagulating basins will start. At the place where the dividing floor begins, an adjustable device will be constructed to open or close the area of the chamber exit to produce some definite loss of head in the water at this point. A loss of head of one to two inches through a slot of uniform width extending from the bottom to the top of the chamber should insure equal amounts of water being added to the upper and lower coagulation basins.

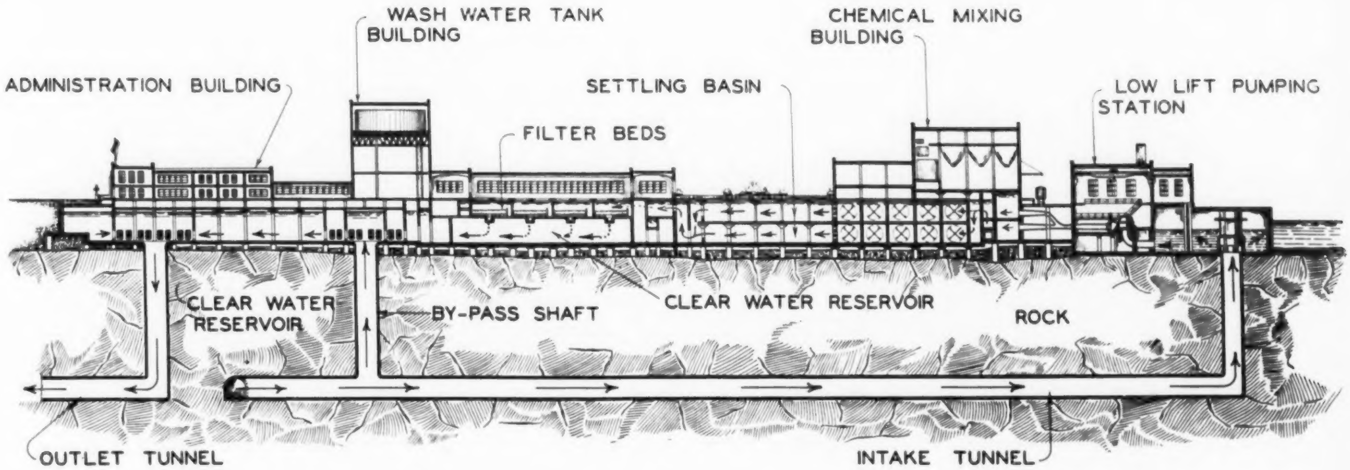

FIG. 4. Cross Section of the Plant

Coagulating Basins

The coagulating basins will be equipped with mechanical agitators fastened to horizontal shafts. The conduits are similar to the around-the-end type of coagulating basin. The water is agitated, but travels forward much more slowly than is general for such basins. The water, therefore, will be traveling in the direction of the center line of the agitating device instead of across the line of the agitators as is customary in the construction of the flocculator type of basin.

The baffle walls are spaced 20 ft. on centers. There are six lines of agitators with the last conduit being more of a distributing conduit to the settling basins than a part of the mixing basins. The water will flow through slots in the wall separating the coagulating basin from the settling basin. Slots will be provided near the bottom, near the center, and near the upper part of the wall. It is likely that a portion of the slots will remain closed during operation. The arrangement of slots is to provide flexibility and so to facilitate changing the level at which the water enters the settling basins should operating experience indicate that a change is desirable.

Settling Basins and Filters

The settling basins are two-story structures with about 16 ft. of water in each story. The water will pass from the coagulating basins through the settling basins to the opposite end, then under a curtain wall and up in a chamber where it may be recarbonated when lime is used in the treatment. The water will pass out of the settling basins through gates into a conduit running along the end of the basins. This conduit connects to the conduits leading to the filters. The water level in the settling basin will be slightly higher than the elevation on the filters. About one-third of each settling basin is to be equipped with continuous sediment-removing equipment.

Each filter unit will be 54 ft. long and 26 ft. wide and have a surface area of 1,390 sq.ft. The top of wash water troughs is to be set about 27 in. above the top of the sand, or at an elevation such that no part of the trough will touch the sand when the filter is in service. The wash water gutter is to extend lengthwise of the filters, along one sidewall. The top of the filter well is elevation plus 17.0. Perforated pipe underdrains, 4 in. in diameter, spaced on 12 in. centers, will be shown on the plans but it is planned to allow bids on other

types of filter bottoms. The manifold is to be concrete extending along under the floor of the filter and tapered from the gallery end to the back of the filters.

Any of the coagulants used in water treatment is satisfactory for Lake Michigan water. The water contains microscopic organisms which tend to clog the filters unless they are removed before the water reaches the filters. The experimental work has shown that the use of small quantities of lime aids the removal of micro-organisms before filtration.

The plant will be equipped for storing, and feeding into the water, aluminum sulfate, ferrous sulfate, ferric sulfate, lime, sodium silicate, sulfuric acid, powdered activated carbon, ammonia or ammonium sulfate, and chlorine.

Substructures

The substructure, the foundation walls and piers of which extend to bedrock, is constructed entirely of reinforced concrete. All earth within the plant limits will be excavated to rock in order that the nature and condition of this rock may be accurately determined before construction begins. The substructure is divided into two portions designated as the east and the west.

The east substructure is approximately 850 ft. long by 480 ft. wide containing the following basins listed in order as they occur from east to west: intake basin, screen chamber, low lift pump room, raw water header basin, chemical mixing basin, coagulating basins, and settling basins. In general, the coagulating and settling portions of the west substructure are divided into three similar basins, each double decked and separated from each other by galleries. These galleries house the machinery for the flocculators and scrapers.

The west substructure is approximately 700 ft. long by 900 ft. wide. It will contain 80 filters grouped in units of 20, each filter having the area of approximately 1390 sq.ft. It will also contain the clear water reservoir which will give a capacity of approximately 50 million gallons.

Design Features

The galleries, which are designed as rigid frame structures, and the retaining walls will be self supporting and capable of resisting any external earth pressures or internal water pressures to which they may

be subjected. The structure is further designed to resist any hydrostatic uplift which might occur when the basins are dewatered.

All design assumptions, unit stresses and types of construction are in accordance with the provisions of the new Chicago Building Code wherever they apply. Because of the nature and purpose of the structure, special attention was given to the concrete plans and specifications as to water tightness, density and surface finish. Approximately 0.45 per cent of temperature or shrinkage steel is

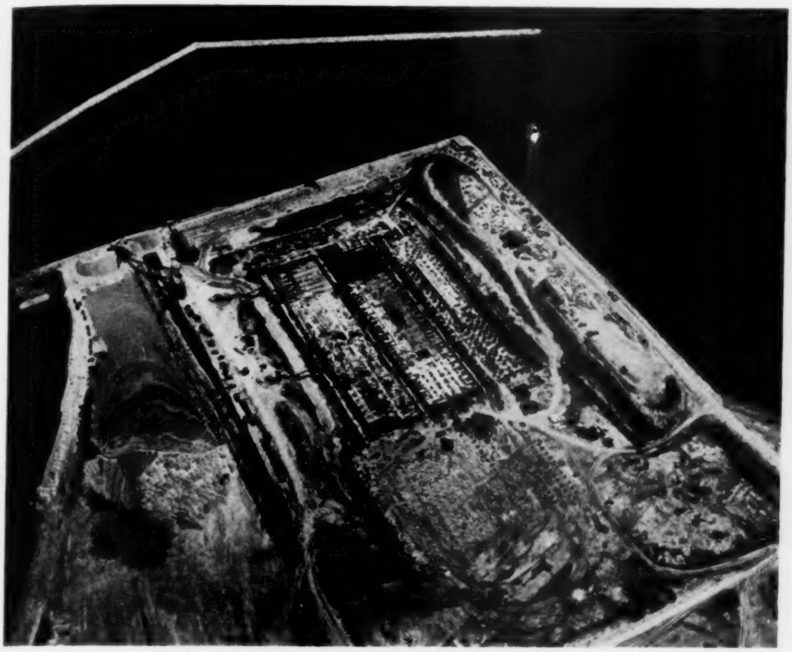

FIG. 5. Panorama of Entire Project After Construction Had Been Started Within the De-Watered Cofferdam

provided in all of the heavy gallery and retaining walls which support heads of water varying up to 35 ft.

Due to the great length and width of the structure, a comprehensive system of expansion joints was developed. In general, these joints are located about 160 ft. center to center in both directions. Special care has been taken in all ways possible to prevent cracking in walls or slabs in the basins, or galleries adjacent to the basins. This is obviously done not only to prevent leakage from the basins but also

to prevent infiltration of ground water or other sources of contamination into the basins.

The prevention of cracking is also particularly essential in structures of this nature in order to safeguard the reinforcement against corrosion and the concrete against general deterioration.

All basins are anchored to bedrock. Flat slab construction with "Tied" columns is used wherever possible as this type of design was found to be the most economical. Panel sizes are generally 20 ft. square.

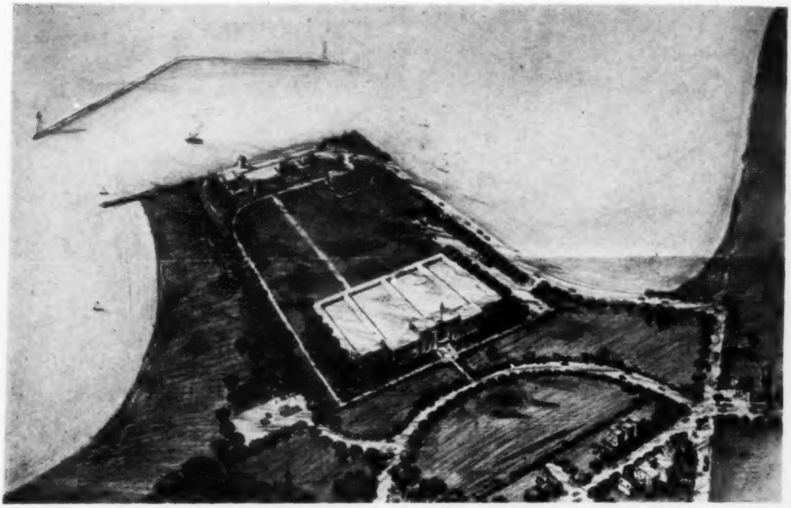

FIG. 6. Architect's Conception of Finished Project

Superstructures

Four superstructures will house the offices, laboratories, chemical and mechanical equipment and supplies necessary for the operation of the plant. They will be of steel skeleton construction with brick or stone walls and reinforced concrete floors. The roof construction will in general be precast concrete tile.

The Administration Building located at the west entrance to the plant will house the general offices, main chemical laboratories, wash water pump room and boiler room. This building has two wings approximately 34 by 100 ft., two stories high. A spacious lobby will connect this building to the filter building, the overall dimensions

of which are about 400 by 600 ft. This building includes 80 filter beds together with their operating galleries.

The chemical building will be located at the east end of the plant over the chemical mixing basin in the substructure. This building will house all chemical feeding equipment and will provide storage space for approximately a 30-day supply of chemicals. The building is served with a switch together with a 40-foot concrete roadway for truck delivery. It is approximately 480 ft. long and has an average width of 60 ft.

Directly to the east of the chemical building is the low lift pumping station and screen house. This building includes the low lift pumps, a boiler room, the electrical equipment, a machine shop and a screen room for servicing the screens in the intake basins. It is approximately 90 ft. wide by 500 ft. long and fronts on Lake Michigan.

The present P.W.A. grant is for 45 per cent of approximately 12 million dollars worth of work which, as stated before, must be completed by March 1, 1941. After the termination of the grant period, the city is obligated to complete the remainder of the South District Filtration Project with its own funds provided that no further P.W.A. assistance is available. It is expected that the Filtration Plant will be put into operation sometime in 1943.

The design and the construction of the South District Filtration Project, are being carried out by the Bureau of Engineering of the Department of Public Works of the City of Chicago, the personnel of which includes: Hon. Edward J. Kelly, Mayor; Hon. Oscar E. Hewitt, Commissioner of Public Works; John P. Wilson, Deputy Commissioner of Public Works; Loran D. Gayton, City Engineer.

Personnel

The design is being carried out under the supervision of the following: O. B. Carlisle, Engineer of Water Works Design; John H. Ryckman, Assistant Engineer of Water Works Design; Wm. F. Martin, Senior Structural Designer; Edwin A. Randall, Senior Mechanical Designer; and John R. Baylis, Physical Chemist.

All the construction is being done by contract under the direct supervision of the following: James J. Versluis, Engineer of Water Works Construction; John Dean, Assistant Engineer of Water Works Construction; Carl G. Riggenbach, Assistant Engineer in charge of Filtration Plant Proper; John A. Eagan, Assistant Engineer—Bulkhead and West Substructure; John M. O'Gara, Assistant

Engineer—Breakwater and Approach Fill; Arthur V. Schutz, Assistant Engineer—Tunnels and Park Fill; W. C. Kenney, Assistant Engineer—In Charge of Office; James W. Pearce, Assistant Engineer—East Substructure.

Discussion by *Arthur E. Gorman.** Engineers of Chicago and particularly those serving the city water works have a right to be proud that "at long last" Chicago has under construction the world's largest filtration plant. A number of interesting features of design and construction pertaining to the South District Filtration Plant have just been described. From them it may be observed that real progress is being made on the project.

Without wishing to detract any proper credit due the engineers who are now engaged in expediting this important public project, I should like, in discussing Mr. Gayton's presentation, to recall and I hope record some of the important contributions made toward this project by persons who were not referred to in the paper.

History records the heroism of a soldier, Arnold Van Winkleried, who at the battle of Sempach, when the Swiss failed to break the serried ranks of the Austrian knights, deliberately sacrificed his life by throwing himself onto the spears of the enemy, thus making a break in their ranks through which a phalanx of Swiss soldiers forced their way to an ultimate victory.

There were Arnold Van Winklerieds in the long drawn-out battle for filtration and metering of Chicago's water supply; and now that a victory has been won, is it not proper that acknowledgement be made of those who contributed to the success of this achievement?

It is especially fitting that this be done at this time, for many of the men living or dead to whose work I wish to refer were members of the Illinois Section of the A.W.W.A.

Anyone who has had experience with the development of a great public improvement knows that, where controversy exists, progress is likely to be slow. Chicago's filtration and metering project was no exception. A particular bone of contention was the metering phase of the plan. In facing this issue it has taken men of real courage to stand up for what they knew to be best for their community, even though metering was politically unpopular. First, I should like to mention four men—not now associated with this project—who gave

* Member, Illinois Section, A.W.W.A.

years of effort to its advancement, often against serious odds, namely: Col. A. A. Sprague, former Commissioner of Public Works, John Ericson and Myron B. Reynolds, former City Engineers, and Alfred H. Marshall, Filtration Designing Engineer. The last three died while in the city's service.

It was Sprague, Ericson and Reynolds who advocated filtration of Chicago's water supply under a system of water conservation which would make it economically sound. It was Al. Marshall who, in my opinion, directed the lion's share of the preliminary design and layout work on the basis of which the present filtration plant is being built. I recall vividly the many meetings with the Chicago Park District engineers from 1934 to 1938 relative to a lake front site for the South District Filter Plant and which resulted in the general layout along present lines. The breakwater, the cofferdam, the park fill, the filter plant approaches, the direct intake, the settling basins with their double deck feature, the filter arrangement and many other items under design and construction today are in substantial, and in some cases almost in exact, accordance with the general layouts made in the Division of Water Purification under Al. Marshall, although in a number of instances the materials of construction differ.

Historically, filtration of Chicago's water supply has been advocated for years, but its acute public health need was made evident by the extensive investigations of the Division of Water Safety Control which was first organized in the Department of Health in January, 1924, following a localized typhoid epidemic on the south side. The effect of these investigations and subsequent reports was the appropriation by the City Council in 1926, on the recommendation of the Commissioner of Public Works, A. A. Sprague, and the City Engineer, John Ericson, of $50,000 for the construction of an experimental filtration plant.

City Engineer Ericson then directed that the organization of the research work and the staff for experimental work on filtration of Lake Michigan water for Chicago be placed in this newly created division, which, in January, 1926, had been transferred from the Health Department to the Bureau of Engineering, Department of Public Works. John R. Baylis of the Montebello Filter Plant at Baltimore was employed in the Division of Water Safety Control in the fall of 1926 and with the assistance of Junior Sanitary Engineer Carl Spear (deceased) of this same division, prepared the preliminary sketches for the present experimental plant on the basis of which,

Mr. Marshall, then in the Division of Water Works Design, directed the detailed design of the plant.

Official approval for the construction of Chicago's South District Filtration Project did not come about as the result of operating this experimental plant and the free dispensing of filtered water, much as this helped in convincing the public of the value of filtered water. It came, rather, as the result of tireless effort on the part of many loyal engineers in the service of the City of Chicago, supported by the City Council, several mayors, commissioners of public works and city engineers, all actively helped by public-spirited civic organizations and individuals.

During the period 1929 and 1930 special consideration was given to filtration of Chicago's water supply under a partially metered system. Detailed designs were even prepared of a 576 m.g.d. plant, this work being done while Mr. Gayton was City Engineer and under the immediate direction of Mr. Marshall.

In July, 1931, under instructions from City Engineer Myron B. Reynolds, studies of filtration for the South District, under a metered system, were begun in the Division of Water Purification. Mr. Marshall and a staff of engineers from the Division of Water Works Design were assigned to the Division of Water Purification to make these studies. The work embraced many items of investigation, among them being: the characteristics of revenue and water consumption by various types of consumers under meters and under assessed rates; the forecasting of population and pumpage for average and maximum days; the sizing of the South District Filtration Plant; layouts and designs at various lake front and inland sites with comparative costs; the preparation, filing and follow-up of the applications to the P.W.A.; and the negotations at Washington which culminated in the contract of August 3, 1938, giving the city a grant of $5,400,000 of federal funds as an aid in financing this project.

In connection with its studies pertaining to meter rates, water consumption and the economics of metering and extension of the distribution system, the Division of Water Purification has enjoyed the fullest co-operation of Bernard W. Cullen, Superintendent, and J. B. Eddy, Engineer of the Division of Water Pipe Extension. The long struggle to obtain a lake front site, the contacts with civic organizations to support this project, the preparation of applications to the Chicago Park District, to the State Waterways Division and to the U. S. Engineer for permission to build the plant on the lakefront,

and the presentation of supporting data were activities carried out by the Division of Water Purification based on the work of Mr. Marshall and his small staff of engineers which consisted of: Maurice A. Drubeck, John J. O'Connor, C. Martin Riedel, Ralph L. Sanders, Frank J. Skall, and John C. Sutphen.

The present project would not be under construction today if it had not been for the whole-hearted support of the Honorable Mayor Edward J. Kelly and the Honorable Oscar E. Hewitt, both of whom actively supported this project and the important legislation necessary to obtain permission to build the plant on the lake front and to sell sufficient water certificates to finance the project to qualify for the federal grant. Much credit is also due the members of the City Council who supported the project at all times and to the many civic organizations whose efforts helped materially in "selling" filtration to the citizens.

In conclusion, let me repeat that in presenting this discussion, I have no wish to detract any proper credit due the engineers who are now expediting the progress of the project. I do feel, however, that since a number of the city employees who had a major part in advancing the project to a point where the contract with the federal government, for financing construction, was accepted, are no longer associated with the project, it is right and proper that a record of their work be made while the story of the project is being presented. In particular when referring to pioneer efforts in the interest of this project, we should not forget the outstanding services of former members of the group who are now dead.

All of us will admit, I am sure, that adequate recognition for work well done makes for a proper *esprit de corps* among public employees. Such recognition serves to mold this intangible but potent force into real public service of the type which is the ultimate goal of all loyal civic employees.

Author's Closure. Mr. Gorman's remarks bring back old memories. When I was a boy we had in the family library a set of books called, *Ridpath's History of the World.* In one of the volumes was a large picture showing the heroic death of Arnold Van Winkleried at the battle of Sempach. It was just as described by Mr. Gorman. But I fail to see the connection between the death of this Swiss peasant, back in the Middle Ages, and the filtration of the Chicago Water Supply in the year 1940.

No blood was shed in bringing about filtration in the City of Chicago. There is ample documentary evidence that the Bureau of Engineering and the people of Chicago have always favored filtration. It was merely a question of securing the necessary funds; when these funds became available we at once started upon the construction of our first filtration plant.

Filtration of the Chicago Water Supply has been discussed pro and con ever since Civil War days, when Ellis Chesbrough, then City Engineer of Chicago, had under consideration the matter of filtering the water supply through the natural sand on the lake shore. This filtration scheme was never carried out but, instead, a tunnel was driven from the original Chicago Avenue pumping station out under Lake Michigan, a distance of two miles, and the original Two-Mile Crib erected at its lake end.

The opening of the main channel of the Sanitary District Drainage Canal, in 1900, and the reversal of the flow of the Chicago River, diverted a great amount of Chicago sewage away from Lake Michigan and greatly improved the Chicago water supply. Although the construction of the Chicago Sanitary District's diversion projects removed the greater amount of Chicago sewage from the lake, there was at the south end of Lake Michigan a rapidly developing industrial district that continued to dump its trade wastes into the Lake, and in addition to these trade wastes, the towns in northern Indiana used the lake as a point of disposal for their domestic sewage. Under certain conditions of wind, and current, this trade waste and sewage was carried into the areas surrounding the intake of the Chicago water supply system.

Conditions became so serious that in 1912, chlorination of the water was adopted at the Four-Mile Crib, and by 1916 the Chicago water supply was being sterilized by chlorination at every one of the pumping stations.

The matter of filtration was once more brought to the front by a somewhat localized typhoid outbreak on the South side in 1923.

In 1924, as Engineer of Water Works Design, I made a study in connection with the proposition of filtering the entire Chicago water supply, and in a report dated January 2, 1925, I submitted my findings and recommendations to John Ericson, who was then City Engineer. In transmitting my report to the City Engineer, I wrote as follows:

Chicago, January 2, 1925.

Mr. John Ericson,
City Engineer.

Dear Sir:

Pursuant to your directions I have made a brief preliminary study in regard to filtering the Chicago water supply. This study was made primarily to ascertain the area of land which should be required for the filtration plants, and how the location of these plants may be affected by changes which are being made in the shore line by the Lincoln Park and South Park Boards.

I have carried the study beyond the original intent by making a rough comparison between the cost of filters and deep water cribs and tunnels.

I do not believe that a system of deep water cribs and tunnels would give any guarantee of clear pure water at all times. For years material has been dumped into the lake from five to thirteen miles from shore and this material may be stirred up by wave action, thus giving a turbid water. Government engineers have found indications that wave action affects the bottom in fifty-five feet of water, and may do so in even deeper water. Also, it is problematical how far from shore the water is contaminated at times of heavy rains and off-shore winds. I believe that a so-called super-tunnel system would be a gamble.

On the other hand, there is absolutely no doubt that filtration would give a satisfactory water supply. The only question to be settled at this time is the location. If they are to be located adjacent to the pumping stations, the necessary land should be secured at once before there is an increase in price. If the filters are to be located in the lake, or at the shoreline, some agreement should be made with the Park Boards so that the necessary areas may be reserved until needed.

This report is preliminary only, and is made in order to call attention to the magnitude of the problem and to show the necessity of giving it consideration at this time. Particular attention is called to the need of installing an experimental filter as soon as possible.

Very truly yours,
(*signed*) LORAN D. GAYTON
Engineer of Water Works Design.

LDG-FG

In the conclusion of the report mentioned in this letter I stated as follows:

"There are many arguments in favor of placing the filtration plants adjacent to the tunnels at the shoreline. Only four large plants would be required. No money would be tied up in lands either for immediate use or for future extensions. The cost of building the plants and the cost of operation should be less than an equal capacity made up of smaller plants.

"There may be some objection to passing the filtered water through long tunnels to the stations in the western part of the City.

"If the filtration plants are placed at or near the pumping stations a great deal of money must be tied up in land for present needs and for future extensions. Eleven separate plants would be required.

"The cost of erection and operation would be higher than for a similar capacity of larger plants.

"The cost of connecting to the present stations would be very great. Each pumping station would have to be remodeled in order to have it operate economically in conjunction with the filter plant.

"I believe that further study will show that it is not practicable to place filtration plants at the pumping stations and that they should be erected near the tunnels at the shoreline."

In a report entitled, "The Quality Problem in Relation to Chicago's Water Supply," submitted to Col. A. A. Sprague, Commissioner of Public Works, by Mr. Ericson, in May, 1925, the City Engineer concurred in my findings and recommendations. As a result of the foregoing we secured an appropriation for the construction of an experimental filtration plant and this plant was built and put in operation in 1928. The purpose of the experimental plant was, of course, to carry out various investigations in connection with the purification of Lake Michigan water in order that we might have definite information upon which to base the economical design and operation of future large permanent plants.

To sum it all up: there has never been any opposition to filtration in the City of Chicago. To filter completely a system supplying approximately 4,000,000 people, and delivering an average of almost one billion gallons of water per day will cost close to $60,000,000. As stated before, the only factor that has held back filtration in Chicago is the necessity of providing this large amount of money.

Experiences With the Palmer Filter Agitator At Olean

By Joseph E. Rehler

THE Olean Filter Plant includes 12 filters, each with a sand area of approximately 11 by 14 ft. Six units were built in the original plant in 1918 and the other six were added when the plant was enlarged in 1929. The later filters were of the same design as those originally built, so that, in effect, the filter design dates from 1918. Although all units were supposed to be practically identical, considerable differences in the strainer system head losses have appeared.

Since no backwash meter was provided, the wash rate, in winter, was set by adjusting the inlet valve to secure a predetermined pressure on the strainer system, usually about $5\frac{1}{2}$ lb. per sq. in. A higher rate resulted in sand being carried out of the filter. In summer the backwash valve was opened wide—giving pressures up to $7\frac{1}{2}$ lb., depending upon the particular filter being washed. Present indications are that this practice gave a rate that may have varied in the individual filters from 2,000 to 3,000 g.p.m., equivalent to 21 to 31 in. rise per min. or 13 to 19 gal. per sq. ft. per min. Filters were washed when the head loss reached 10 ft. or at the end of a 70-hour run. Ordinarily the full 70-hour run was secured and only occasionally, for short periods in summer, was it reduced to from 30 to 35 hr. In all cases the duration of wash was 7 min.

The usual troubles, including mud balls, gravel hills, hard spots, cracks and pulling away from the walls, were experienced, but there was considerable variation in the condition of the units. In winter activated carbon was likely to appear suddenly in the effluent. In the summer, when a filter was drawn down, the algae odor was very strong. Also, in summer, when pre-chlorination was most needed,

A paper presented on June 7, 1940, at the New York Section Meeting, Ithaca, N. Y. by Joseph E. Rehler, Superintendent of Water & Sewerage, Olean, N. Y.

an objectionable taste resulted if the residual entering the filters exceeded about 0.04 p.p.m. An attempt at super-chlorination by the writer's predecessor (the late S. D. Poarch) was not successful from a taste standpoint, due, it is now believed, to dirty filter sand.

Although the maximum filter rate was about 1.5 g.p.m. per sq. ft., 10 ml. samples of the filter effluent showed positive presumptive coliform tests several times per month, indicating that organisms were passing the filters. Often these organisms were resistant to chlorination and appeared in the final treated water. Doubling the pre-chlorine residual and increasing the alum dosage to a minimum of 1 g.p.g. were helpful but did not solve the problem.

Believing, as does Wolman, that progress lies in the direction of spending less time trying to show that the organisms passing the filters are not of the coliform group and more time trying to keep all organisms from the filter effluent, the staff cast about for a method of overcoming filter troubles. Also, it was desired to clean the sand preliminary to another attempt at super-chlorination.

Various Remedies Proposed and Attempted

The usual remedies of hand raking, breaking up hard spots with home-made jets, cleaning with caustic soda were tried and cleaning with sulfur dioxide was considered; but all of the remedies appeared to be palliatives rather than cures and all were open to objections.

The next thought was to increase the backwash rate in the hope that if the sand were once cleaned with chemicals, a higher rate would keep it clean; but this, too, was impractical because the underdrains, which were imbedded in concrete, would not carry more water—nor would the existing backwash pump supply a greater rate. Also there was considerable doubt whether high wash rates alone would keep filter sand clean. Indications seemed to point to some form of agitation as the solution and it was felt that air wash should be tried.

Filter Agitator Tried

At about this time the "Palmer" filter agitator was placed on the market. One was secured for a trial installation. This trial agitator was connected, temporarily, with a fire hose. First the filter was washed in the usual manner until the wash water became clear. Then the agitator was started. The dirt that literally rolled out of the sand was a revelation. For 29 minutes there was no perceptible

clearing and at that time the washing had to be stopped for lack of water.

Results of Trial

As a result of this trial, agitators were placed in all 12 filters, installation being completed in January, 1940. At about the same time a backwash meter was installed for accurate measurement of both quantity and rate of wash.

For the first four months of operation, emphasis was placed on cleaning the sand, rather than on reducing the quantity of wash water. As only enough wash rate is required to place the sand in suspension, however, it was found that the rate could be reduced to about 1,800 g.p.m., a reduction of approximately 25 per cent. In addition, duration of wash has now been reduced to 6 minutes, giving a total wash water reduction of about 35 per cent. There is some indication that a further slight reduction can be effected and as yet no attempt has been made to lengthen the filter runs.

The filters are now free from mud balls and there is no longer the layer of mud, formerly noticeable, on top of the sand after washing. The sand is clean in appearance, having lost its dark muddy color.

The corners of the filters became clean more slowly than the main body of sand, so a home-made perforated-pipe water jet was used to help break up the worst areas. At present practically all of the corners are soft when tested with a rake and are as clean in appearance as the rest of the filter.

Last year, sand from 10 filters was cleaned with caustic soda at about the same time as the trial agitator was secured. The remaining 2 filters were left untreated to see if the agitator alone would clean them. At the present time, no difference can be detected between the caustic-treated sand and that not treated.

No trouble has been experienced recently from carbon passing through the filters but this is not considered conclusive as the greatest difficulty is encountered in winter. Similarly any improvement in bacteriological quality of the filter effluent is as yet inconclusive.

Installation and Operation

A word concerning the installation and operation of the agitators may not be amiss. There is one agitator in each filter, suspended from a bearing at the center and located about an inch above the normal sand level. During washing it rotates at a speed of 9 to 10

r.p.m., driven by the reactive force of the water issuing from the jets. The quantity of water passing through the agitator is approximately 0.5 p.p.m. per sq. ft. of filter area. During the backwash an observer can see a rolling action which appears to have a scouring effect on the sand. There is a jet located at each end of the agitator arm and the impulse from these end jets can be felt clearly in the corners of the filters, a distance of approximately 40 in. It is this impulse that apparently accounts for the elimination of hard spots in the corners.

When backwashing a filter, the agitator is first started to break up the sand surface. Then, after about a minute, the backwash water is admitted. At the end of the wash, the agitator is shut off before shutting off the wash water. This insures a smooth and level sand surface for the next filter run.

It is the author's opinion, based on his experience to date, that this filter agitator fills a long-felt need in filter operation.

Protective Coating for Steel Water Lines

By Deming Bronson

THE phenomenal growth of the use of steel pipe for water lines in the past five years, and their inevitable wider use, make the subject of protective coatings one of immediate practical interest. The observations in this paper shall be limited to the commercially practicable and available types of interior and exterior protection for steel water pipe without consideration of any theoretical or idealized protection which might be devised if the economics of pipe protection did not intrude. The subject logically divides itself into two parts—interior protection and exterior protection—and these will be discussed in turn. It is appropriate, also to discuss briefly the theory and practice of the selection of interior and exterior pipe protection for existing conditions, although necessarily such discussion will have to be more or less generalized.

Any engineer who has had experience with protective coatings can design a form of protection for steel water pipe which would satisfactorily take care of any and all conditions. When he finished with his design, devised the means of application of the several elements of the protection and added up his total costs, however, he would find that he had an idealized protection which would be commercially impracticable. In other words, it is one thing to plan and design a theoretically ideal protective coating and quite another to achieve it in an economical and practical manner. The question of steel pipe protection is essentially an economic one and this factor should not and cannot be overlooked.

Perhaps the simplest and most well-known form of interior protection for steel water pipe is galvanizing. Thousands of miles of galvanized steel pipe have been installed annually for a great many

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years, both for water service lines and for the smaller distribution mains. Briefly, galvanizing is the application of zinc in molten form to steel pipe to accomplish a deposition of the zinc on the interior walls of the pipe. In this process, the pipe is pickled in acid, washed and neutralized in water, and dipped in a fluxing bath. It is then heated and rolled into a molten bath of zinc which is maintained at over 800° F. In this the pipe is allowed to remain until it acquires the temperature of the molten zinc. The weight of the galvanizing usually averages two ounces per square foot of surface coated. One prominent manufacturer of galvanized steel pipe supplements this process with a special chromate treatment to resist discoloration and formation of what is called white rust. It is claimed that such additional treatment preserves, for a considerable time, the smooth glistening surface or metallic lustre of the hot galvanizing.

Two steel pipe manufacturers offer cement lining. The type of cement used is usually of low lime and high silica content, having low solubility properties. The mix is applied by a centrifugal method and is followed by a curing treatment that increases the strength of the cement and reduces the inherently low shrinkage factor. This type of lining has substantial acceptance for special installations, particularly for hot water lines in building construction.

A third form of interior protection for steel water pipe is the application of two coats of a cold-applied liquid coal-tar solution. Only a two-coat job is recommended or offered by leading applicators because the double coat removes the possible existence of pin holes or voids in a single coat. This liquid coal-tar coating, which is maintained in liquid form by use of coal-tar solvents, is usually applied by a combination of spraying and brushing. After the first coat is dry, the second coat is applied similarly, and is then allowed to dry before shipment of the pipe is made from the pipe mill. The resultant thickness of these two coats of cold-applied coating is about $\frac{1}{64}$ inch. The surface is black and glossy and, with the interior surface of the pipe properly prepared by removing rust and mill scale, the surface is smooth, hence increasing the flow of water through the pipe during the life of the lining.

There is still commercially available, in the industry, pipe that has been dipped in a hot asphalt bath. This dipping is accomplished with the pipe either in horizontal or vertical position. The

pipe is made clean of loose scale, rust, dirt, oil and grease—the better practice includes the heating of the pipe until it is perfectly dry—then dipped into a tank of hot asphalt and allowed to remain until the pipe and the asphalt reach practically the same temperature. The pipe is then withdrawn, and, if horizontal dipping is used, is held at a sufficient angle to allow any surplus coating to drain off naturally without wiping or swabbing. Asphalt-dip lining is from .01 to .03 inch thick.

Coal-Tar Enamel for Interior Protection

The most generally accepted and most highly regarded form of interior protection for steel water pipe is hot spun coal-tar enamel. In this process the interior surface of the pipe is made clean and free of oil, grease, rust, and loose mill scale and is then primed with a cold-applied liquid coal-tar primer. This primer is applied by spraying or brushing or by a combination of both. The primer is a bonding medium between the metal and the hot-applied enamel. After it is properly dry, the hot coal-tar enamel is applied from a trough, from a weir, or from a retractable feed line while the pipe is revolving on its own axis at a relatively high peripheral speed. The force of revolution of the pipe distributes the enamel evenly over the inner pipe wall in the desired thickness, as controlled, within close limits, by various means depending upon the mechanical devices used in application. The revolving of the pipe is usually continued (with or without the introduction of cooling water on the inner surfaces) until the enamel has cooled to a non-sagging temperature or has set to firmness. The resultant surface of the hot-applied enamel lining is black, smooth and glossy and provides the utmost in increased flow capacity. It is usually installed $\frac{3}{32}$ plus or minus $\frac{1}{32}$ inch thick.

Lining—Manufacturer's Job

In the foregoing brief outline of commonly used types of interior pipe protection which are commercially available, no attempt has been made to differentiate between good, bad, or indifferent methods of application. These conditions can, of course, exist in this work just as they do in any other manufacturing process. Suffice it to say that the amount of interior protection of steel pipe that is applied outside of the pipe mills is negligible. Those interested in this type of work have found that the means for obtaining

proper interior pipe surfaces and the need for rather elaborate and costly equipment limit the place of application to the pipe mills or, in some cases, to pipe fabricators' yards. Practically no lining of steel pipe is done by the consumers of the pipe themselves because it is difficult and costly to accomplish suitable operating conditions for relatively small consumption of pipe. In other words, interior pipe protection has come to be essentially a factory-processed job.

Exterior Protection

Exterior protection is a more complex and more difficult subject. A simple method is the application of red lead and/or aluminum paint. This type of protection is confined to pipe which is not to be buried underground but which will be exposed to the elements. For such service, even the most enthusiastic advocates of bituminous coatings yield to this or an equivalent form of paint protection for two reasons: first, the paint materials seem to maintain their protective value better, under such outdoor exposure, than do bituminous coatings; and second, the cost of such a protective paint coating is usually lower than that of the bituminous coatings necessary for underground steel pipe protection.

Another form of exterior protection for steel water pipe is galvanization. The method of application of galvanizing to the exterior of pipe is the same as that described above for application to the inside of steel pipe. It is obvious, of course, that while exterior galvanizing alone can be applied, it is impossible to apply interior galvanizing without at the same time galvanizing the exterior. On small pipe where the cost is not excessive per diameter inch and for exposed service or for underground service in certain soils, galvanizing has established a place for itself as an exterior protection.

When the interior protection of steel water pipe is sought by the use of hot asphalt dip, it is, of course, necessary simultaneously to coat the outside of the pipe. The deposit of asphalt on the exterior of the pipe, like the dip on the interior, will run from .01 to .03 inch in thickness. After the pipe is cooled, a coat of whitewash is usually applied to the coating to prevent the pipe from sticking together and from absorbing heat from the sun while in transit or while exposed in the field before laying.

An exterior protective asphalt coating is also available in combination with wrappings of one kind or another. Both organic and inorganic materials may be employed, and they may be applied in one

or more layers. Muslin fabric soaked in the hot dipping compound just prior to application, or mesh fabrics which have been impregnated with asphalt are used to a decreasing extent. More commonly used is an asphalt-saturated rag or asbestos felt of suitable width for smooth spiral application. This impregnated felt weighs about 14 lb. per 100 sq.ft. and is applied under tension in such a way that the felt is smoothly and snugly bonded to the underlying asphalt. It is an interesting observation that, when asphalt has been selected as the main line of defense in exterior protection for steel water lines, the greater demand is for the use of asphalt-impregnated rag felt rather than asphalt-impregnated asbestos felt. This is largely the result of the desire for economy in the selection of the reinforcing wrappers rather than a general preference for the rag felt as such. In the application of asphalt coating-and-wrapping, there is again the obvious prevalence of mill application rather than field application. This is, undoubtedly, because few consumers of steel pipe can justify the investment in the mechanical equipment required to obtain a satisfactory job of application.

Unlike the cast-iron pipe practice, there is practically no steel pipe purchased with coal-tar dip. Very rarely, however, a job of this kind is purchased and then it is selected by reason of the necessity of providing man-hours of relief labor for application of heavier protective coatings in the field. When this course of action is taken, the resultant job is a tribute to that necessity rather than to the attainment of a thoroughly effective exterior protection.

Coal-Tar Enamel for Exterior Protection

The most widely used form of exterior protection for steel water lines is coal-tar enamel, with or without a reinforcing wrapper. Significantly enough in the water works industry, there is practically no use of rag-felt wrappers in connection with coal-tar enamels. The preponderant preference is for coal-tar-impregnated asbestos-felt wrappers. In commercial practice this combination of coal-tar enamel and asbestos-felt wrapper is again accomplished in the pipe mills, or in equivalent manufacturing establishments, where the mechanical equipment can be used advantageously by spreading its investment over a large footage of pipe.

In the larger sizes of steel pipe, asbestos felt is less often used than in the smaller sizes. The line of demarcation here appears to be determined by the facilities for handling the pipe in the field during

its installation. This does not mean that it would not, in most cases, enhance the value of the coal-tar protective enamel on large pipe to have it shielded and reinforced by an asbestos-felt wrapper; rather, it means either that the economies of the design of a large-diameter steel pipe do not allow for the cost of application of the asbestos-felt wrapper, or that other means (such as sand backfills) are used in the place of shielding-felt wrappers as a safeguard against distortion of the exterior protective coating through soil stress in the ditch. With the smaller sizes of steel water pipe, however, there are low-cost mechanical methods of application of the felt which have encouraged the use of an asbestos-felt wrapper because its applied cost is so much less on the small pipe, in proportion to size, than it is on large pipe. When coal-tar enamels are thus reinforced by bonded asbestos-felt wrappers, sand backfills need not be used; and in this way the construction cost elements are balanced.

It is the usual practice among commercial applicators to apply a coating of whitewash or light-reflecting kraft paper over the previously mentioned exterior coating, or coating-and-wrapping, to prevent the pipe from sticking together in transit and to reflect the heat from the sun during exposure before laying.

Experience in Other Fields

Since the selection of materials in the design and construction of any structure necessarily involves a knowledge of conditions under which that structure is to exist, and at the same time involves consideration of the cost of such structure, it is impossible to lay down any rules that can be called "the best practice" universally. In the matter of interior and exterior protection of steel water pipe, there are, however, certain well established and generally recognized principles which can serve as a guide in the selection of steel pipe protection. Most of these principles can with safety be adopted from the experience of others in similar fields of construction. The water works industry is fortunate that other industries, which long ago adopted the use of steel pipe almost exclusively, have already proved certain fundamentals which are available for the cost of reading. The American Gas Association and the American Petroleum Institute have conducted independent and exhaustive field burial tests and the final disinterment of the latter's test samples is being made this summer. Their previous examinations of a series of samples of

protected steel pipe have already established the following basic principles:

(1) Protective coatings, to be economically justifiable, must have substantial thickness.

(2) In general, the coal-tar enamel coatings are more stable and less susceptible to absorption of water than are asphalt coatings.

(3) The use of asbestos-shielding wrappers so greatly increases the service life of protective coatings that their use gives a return on the investment far in excess of their cost.

(4) No protective coating, however it is constituted, can give effective and durable protection unless it is properly applied to steel pipe surfaces that are in proper condition to receive them.

Factors in Selection

In the water works industry, as in the gas and oil industries, there are undoubtedly situations in which it is unnecessary, or even economically unsound, to make an investment in interior protection, or exterior protection, or both. It should be remembered that the primary purpose of interior protection on steel water lines is not to prevent destructive corrosion of the steel from the inside; on the contrary, it is meant to provide and maintain a higher flow capacity through the elimination of tuberculation, incrustation, and sedimentation. If a given water in a steel pipe line is known to be non-tuberculating, not provocative of incrustation, and does not deposit sedimentation on the walls of the pipe, then interior pipe protection may not be justified. On the other hand, if tuberculation, incrustation, or sedimentation are known to exist, or tests of the water show that these deteriorating effects are likely to occur, interior pipe protection is not only justified but its use is demanded from the engineering and economy viewpoints.

The selection of an interior protection for steel water lines should, in most cases, be made with the purpose of permanently maintaining high flow capacity. To get this permanence the lining selected should be as non-absorbent of water as possible, as it will constantly be exposed; and it should be a lining of substantial thickness to endure. The present-day thought on material having these two qualities is that hot-applied coal-tar enamels most nearly approach the ideal.

The selection of exterior protection for steel water lines is similarly a matter of judgment and economies. If a new water line is to be

installed in soils which past local history has satisfactorily demonstrated to be non-corrosive, it is certainly unwise to invest money in unwarranted exterior pipe protection. By the same token, the oil and gas industries have demonstrated to their satisfaction that it is not economical to use steel itself as a corrosion protection by increasing the wall thickness of the pipe beyond that which will adequately carry the pressures with the desired factor of safety. The modern steel pipe with its high tensile strength and concomitant flexibility does not require excessive wall thickness to serve its designed function, and it is uneconomical in this day and age to attempt to provide corrosion protection by such a method. Furthermore, it has been demonstrated in the report of "Steel Corrosion Studies of 1934" (U. S. Bureau of Standards Research Paper RP883) by K. H. Logan, dealing with the rates of loss of weight and pitting of ferrous specimens, that the addition of copper, alone, to steel does not increase its resistance to the action of most soils.

In the third paragraph of the summary of that same report, a worth-while warning note is sounded against too general acceptance of what might be thought of as a non-corrosive soil condition.

"There is a relation between the average rate of corrosion of iron in soil and the soil type, but the dispersion of the data which makes up the average rate is very large. This is because conditions within a soil type are not always the same. It follows that while a soil may be designated as non-corrosive, a pipe in that soil may develop a single leak within a few years. . . ."

Good Practice and Specifications

It is this ever-prevalent risk of costly destructive corrosion which justifiably directs the elimination of such chances by the adoption of a policy of "playing it safe." And to play it safe within the bounds of economic good sense, means to employ exterior protection on steel pipe whenever and wherever judgment or experience indicate a fair opportunity for return on the additional investment. If the selector of an exterior protection for steel pipe will follow the experience of his predecessors whose working conditions are, in the broad sense, the equal of his own working conditions, then suitable exterior protection for given conditions will be made. Here, again, thought should be given to the established principles of substantial thickness, proper application, and adequate reinforcement.

In the preparation of the "Tentative Specifications for Coal-Tar Enamel Protective Coatings," (Jour. A. W. W. A. (Jan. '40))* full and complete thought has been given to the practical viewpoint of commercial availability in relation to good practice. While the steel pipe industry, the protective coating industry, and studious engineers are conscious of the fact that the last word in steel pipe protection has not yet been reached—and probably never will be reached—there is an astonishing unanimity of thought in the water works industry that these specifications provide sound and adequate practice for such protection. No industry and no given field of activity can withstand the march of progress. It follows that, as time goes on, improvements are bound to be made in the design of protection described in the specifications. Until these progressive improvements are brought to light and their worth and value demonstrated, it is perhaps not unreasonable to suggest, nor is it improper to forecast, the selection of this recommended "good practice" for the protection of steel water pipe.

* These documents became standard specifications approved by the Board of Directors of the A. W. W. A. on April 25, 1940. They are designated as 7A.5—1940 and 7A.6—1940 and are now available as a reprint under one cover.

Typhoid in Large Cities of the United States in 1939

As IN the past years, the information used in the compilation of this report was secured by addressing a communication to the health officer of each of ninety-three cities, requesting the number of deaths from typhoid, both among residents and among nonresidents, which were recorded in 1939. As the population figures of the 1940 census are not yet available, the uncertainty of rates based on local estimates is recognized. In all instances the local estimate when furnished by the health officer has been used. With the approach of the new census day there are some who apparently feel that they have been unduly optimistic in claiming population increases and now submit estimates below that of 1938. The total of each of two groups of cities (South Atlantic and East North Central) is below that of last year. Probably no grave statistical error has been committed by using the local estimate rather than falling back on the 1930 census figures. In fact, any such error is insignificant when compared with the variety of figures on typhoid deaths which are submitted (through different channels) by the same local health department or the state health department. All re-allocations by place of usual residence or by source of infection are not completed at the time this report is prepared. There are some cities and states in which re-allocations are apparently never prepared. There is great need of uniformity in this respect. It is expected that after the 1940 census figures become available this report can be revised, but even then it is anticipated that the corrections will be of only a minor character.

Reprinted by permission, from the Journal of the American Medical Association (Vol. 114, pp. 2103-2107, May 25, 1940). Previous reproductions of this annual summary have appeared in the Journal of the American Water Works Association as follows: Vol. 20, p. 257; Vol. 21, p. 963; Vol. 22, p. 1122; Vol. 23, p. 1059; Vol. 24, p. 1066; Vol. 25, p. 1157; Vol. 26, p. 939; Vol. 27, p. 1593; Vol. 28, p. 1123; Vol. 29, p. 1177; Vol. 30, p. 1456; and Vol. 31, p. 1561.

Paratyphoid has again been excluded. In Tables 1 to 8 inclusive (as well as in Table 10) a special note has been made of cities in which all deaths occur among nonresidents. This practice was applied first to the figures for 1937 and has been continued through 1939. Another symbol has been used to indicate those cities in which more than one third of the reported deaths were stated to have been among nonresidents.

Special attention is called to the cities, listed in Table 9, which

TABLE 1

Death Rates of Fourteen Cities in New England States from Typhoid per Hundred Thousand of Population

	1939	1938	1937	1931–1935	1926–1930	1921–1925	1916–1920	1911–1915	1906–1910
Bridgeport	0.0	0.0	0.0	0.3	0.5	2.2	4.8	5.0	10.3
Fall River	0.0	0.0	0.0	0.2°	2.2	2.3	8.5	13.4	13.5
Lynn	0.0	0.0	0.0	0.2	1.5	1.6	3.9	7.2	14.1
New Bedford	0.0	0.0	0.0	1.1°	1.5	1.7	6.0	15.0	16.1
Cambridge	0.0	0.0	0.8	0.9	2.1	4.3	2.5	4.0	9.8
Lowell	0.0	0.0	1.0	1.0°	2.6	2.4	5.2	10.2	13.9
Worcester	0.0	0.5*	0.5*	0.6	1.0	2.3	3.5	5.0	11.8
Hartford	0.0	0.5*	1.1	1.2	1.3	2.5	6.0	15.0	19.0
Springfield	0.0	0.7*	0.7*	1.0	0.4	2.0	4.4	17.6	19.9
Waterbury	0.0	0.9	0.0	0.4	1.2	1.0	8.0	18.8	—
Somerville	0.0	1.0	0.0	0.4	1.3	1.6	2.8	7.9	12.1
Providence	0.4	0.0	0.4	1.1	1.3	1.8	3.8	8.7	21.5
Boston	0.4†	0.6†	0.4*	0.6	1.2	2.2	2.5	9.0	16.0
New Haven	1.2†	1.2†	1.2	0.7	0.6	4.4	6.8	18.2	30.8

* All typhoid deaths were stated to be in nonresidents.

° Rate computed from population as of April 1, 1930, as no estimate for July 1, 1933, was made by the Census Bureau.

† One third or more of the reported typhoid deaths were stated to be in nonresidents.

report no typhoid death during the past two or more years. Bridgeport heads the list with no death in six years. Fort Wayne reports no death in five years, South Bend and Utica no death in four years, Fall River, Lynn, Milwaukee, New Bedford and Wichita no death in three years. Attention is also directed to the ten cities (Fort Worth, Scranton, Louisville, Norfolk, Canton, San Diego, Dallas, San Antonio, Camden, El Paso) which were not placed on the honor roll in Table 10 merely because they have been charged with deaths among nonresidents. These cities may appropriately be considered

as belonging in the honor group. Some cities are far more liberal than others in their attitude in accepting for hospitalization cases of communicable disease from the neighboring rural areas. In some instances legal circumstances compel the city to accept cases from the county.

Eleven of the large New England cities (there were only seven in 1938) report no death from typhoid in 1939 (Table 1). Bridgeport has extended its good record to six years. Fall River, Lynn and New Bedford record no death in three years; Cambridge and Lowell, none in two years. Somerville, after passing through four successive years without a death, followed by one death among residents in 1938, now returns to the honor roll with no death in 1939. Springfield reports no death among residents during the past five years, Worcester none for four years, Hartford none for two years. Of three deaths reported from Boston, two occurred in nonresidents. There were only three deaths among residents in the fourteen cities in the New England states (one each in Boston, Providence, New Haven)—a truly remarkable record. The New England cities as a whole (population 2,657,824) have regained first place among the grouped cities (Table 13). This enviable place was lost in 1938 to the cities of the East North Central states, which now find themselves in third place in 1939. The Middle Atlantic cities are in second place. In the New England cities there were recorded six deaths in 1939, only one half the number for 1938 and 1937.

The Middle Atlantic states (Table 2) have a group rate (0.37) which is lower than that of 1938 (0.44) and 1937 (0.51). There has been no death recorded in Utica for the past four years, none in Elizabeth for two years. Eight cities (Utica, Elizabeth, Syracuse, Paterson, Yonkers, Jersey City, Albany, Trenton) report no typhoid death in 1939. There were six such cities in 1938, seven in 1937. Scranton records no death among residents during the past six years, Camden none for two years. Buffalo, after two years with no death among residents, reports one such death in 1939; Erie, after no death among residents in four years, reports two such deaths in 1939. Newark records the highest incidence of typhoid in twelve years (twenty-five cases) and the largest number of deaths in seven years (seven deaths). It is stated that practically all of the cases were due to contact with a mild case acting as a carrier. New York reports twenty-two deaths (the same number as in 1938), of which twenty

(as in 1938) were among residents. Philadelphia reports twelve deaths (fifteen in 1938), six among residents. Pittsburgh reports three deaths, two among nonresidents. After two years with no death, Reading reports one death among residents. The health officer of Scranton records no typhoid death; however, the Pennsylvania Department of Health has charged one death to this city. Syracuse reports no death among residents during the past four years.

TABLE 2

Death Rates of Eighteen Cities in Middle Atlantic States from Typhoid per Hundred Thousand of Population

	1939	1938	1937	1931–1935	1926–1930	1921–1925	1916–1920	1911–1915	1906–1910
Utica	0.0	0.0	0.0	0.6	1.1	3.9✱	—	—	—
Elizabeth	0.0	0.0	0.8	0.9	1.6	2.4	3.3	8.0	16.6
Syracuse	0.0	0.5*	0.0	0.8	0.8	2.3	7.7	12.3	15.6
Paterson	0.0	0.7	0.0	0.9	1.0	3.3	4.1	9.1	19.3
Yonkers	0.0	0.7	0.0	0.7	0.5	1.7	4.8	5.0	10.3
Jersey City	0.0	0.9	0.3	0.2	0.9	2.7	4.5	7.2	12.6
Albany	0.0	1.4	1.3†	1.1	1.8	5.6	8.0	18.6	17.4
Trenton	0.0	2.4†	0.8*	1.1	2.1	8.2	8.6	22.3	28.1
Buffalo	0.2	0.0	0.2*	0.6	2.7	3.9	8.1	15.4	22.8
New York	0.3	0.3	0.3	0.8	1.3	2.6	3.2	8.0	13.5
Pittsburgh	0.4†	0.9†	0.7†	0.9	2.4	3.9	7.7	15.9	65.0
Rochester	0.6†	0.3	0.0	0.4	1.7	2.1	2.9	9.6	12.8
Philadelphia	0.6†	0.7¶	1.4	0.9	1.1	2.2	4.9	11.2	41.7
Scranton	0.7*¶	0.0¶	0.7*¶	1.4	1.8	2.4	3.8	9.3	31.5
Reading	0.9	0.0	0.0	0.4	1.6	6.0	10.0	31.9	42.0
Newark	0.9	0.4	0.0	0.3	0.9	2.3	3.3	6.8	14.6
Erie	1.6	0.0	0.8*	1.0	0.9	2.3	6.9	49.0	46.6
Camden	2.5*	1.6*	1.6	2.8	4.4	5.9	4.9	4.5	4.0

*† See footnote Table 1.

✱ Incomplete data.

¶ Typhoid deaths furnished by Pennsylvania Department of Health, Harrisburg.

In the group as a whole (population 13,602,500) there were fifty-one deaths in 1939 compared with fifty-nine in the preceding year. There was but one geographic area which had a lower death rate for 1939, the New England.

The rate (Table 3) for the South Atlantic cities (1.03) is much improved over the rates of 1938 (1.74) and 1937 (1.96) and is considerably lower than that of 1936 (1.55). It is but slightly more than

a third the rate for the quinquennial period 1931–1935 (2.70). In these cities (population 2,622,237) there occurred twenty-seven deaths in 1939 and forty-six in 1938. There was but one city (Wilmington) without a death in 1938, although Norfolk recorded its one death among nonresidents. Nine (one-third) of the deaths in the group of cities as a whole occurred among nonresidents. The percentage of deaths among nonresidents is less significant than for former years. Of seven deaths recorded for Atlanta, two were among nonresidents; of four in Washington, one was among nonresidents. Baltimore records but one death among residents (five among nonresidents). The health officer states that only twenty-four cases

TABLE 3

Death Rates of Nine Cities in South Atlantic States from Typhoid per Hundred Thousand of Population

	1939	1938	1937	1931–1935	1926–1930	1921–1925	1916–1920	1911–1915	1906–1910
Wilmington	0.0	2.7	1.8	1.5	3.1	4.7	25.8 ⁂	23.2 ⁂	33.0
Washington	0.6	1.0†	1.9	2.6	2.8	5.4	9.5	17.2	36.7
Baltimore	0.7†	1.5†	1.2	1.4	3.2	4.0	11.8	23.7	35.1
Norfolk	0.8*	0.8	0.8	4.2	2.2	2.8	8.8	21.7	42.1
Tampa	1.0	2.0†	0.0	3.0	3.8	19.1	43.9 ⁂	—	—
Miami	1.4	3.6†	6.3	2.2	3.5	—	—	—	—
Richmond	1.6	2.7†	3.2†	2.5	1.9	5.7	9.7	15.7	34.0
Jacksonville	1.9	5.2	4.0	1.7	4.4	—	—	—	—
Atlanta	2.2	0.9†	1.9†	7.2	11.1	14.5	14.2	31.4	58.4

*† See footnote Table 1.

⁂ Incomplete data.

(less than half of any previous year) were reported in Baltimore. The three deaths in Richmond, three in Jacksonville, two in Miami and one in Tampa occurred in residents.

The East North Central cities (population 9,883,376) experienced a setback and dropped from first place in 1938 to third in 1939 (Table 4). The number of typhoid deaths increased from thirty-five to forty-four (the rate from 0.35 to 0.44). The rate remains, however, below that of 1937 (0.62) and of the quinquennial period 1931–1935 (0.75). As in 1938 there are seven cities (Fort Wayne, South Bend, Milwaukee, Akron, Evansville, Flint, Dayton) which report no death in 1939. They are not, however, altogether the same cities. Grand Rapids with one death among residents in

1939, Canton with one among nonresidents, Youngstown and Peoria each with two deaths among residents, have been dropped from the honor list. There are four cities (Cleveland, Columbus, Cincinnati, Indianapolis) in which one third or more of the reported deaths were stated to be among nonresidents. Fort Wayne reports no typhoid death in five years, South Bend no death in four years, Milwaukee no death in three years. Chicago reports nine deaths, all among residents. Of five deaths each in Cincinnati and Cleveland, two

TABLE 4

Death Rates of Eighteen Cities in East North Central States from Typhoid per Hundred Thousand of Population

	1939	1938	1937	1931–1935	1926–1930	1921–1925	1916–1920	1911–1915	1906–1910
Fort Wayne	0.0	0.0	0.0	2.2	4.2	12.9	7.3	—	—
South Bend	0.0	0.0	0.0	0.7	—	—	—	—	—
Milwaukee	0.0	0.0	0.0	0.2	0.8	1.6	6.5	13.6	27.0
Akron	0.0	0.4	1.6†	0.8	1.5	2.4	10.6	21.0	27.7 ⁎
Evansville	0.0	0.9*	2.7†	1.9	6.2	5.0	17.5	32.0	35.0
Flint	0.0	1.2	3.0†	0.7	1.6	4.6	22.7	18.8	46.9
Dayton	0.0	1.3†	1.4†	0.8	1.9	3.3	9.3	14.8	22.5
Chicago	0.3	0.3	0.3	0.4	0.6	1.4	2.4	8.2	15.8
Toledo	0.3	0.3*	1.2*	1.3	3.0	5.8	10.6	31.4	37.5
Detroit	0.5	0.2†	0.3	0.6	1.3	4.1	8.1	15.4	22.8
Cleveland	0.5†	0.3†	0.5	1.1	1.0	2.0	4.0	10.0	15.7
Grand Rapids	0.6	0.0	1.0*	0.2	1.0	1.9	9.1	25.5	29.7
Canton	0.9*	0.0	0.0	0.9	1.4	3.3	8.9	—	—
Columbus	0.9†	0.3	1.5	2.0	2.1	3.5	7.1	15.8	40.0
Cincinnati	1.0†	0.6	1.3†	1.4	2.5	3.2	3.4	7.8	30.1
Youngstown	1.1	0.0	1.1	1.1	1.1	7.2	19.2	29.5	35.1
Peoria	1.6	0.0	1.7*	0.9	0.2	3.7	5.7	16.4	15.7 ⁎
Indianapolis	1.9†	1.3	1.3	1.2	2.7	4.6	10.3	20.5	30.4

*† See footnote Table 1.

⁎ Incomplete data.

were among nonresidents. Of eight deaths in Detroit, one was among nonresidents. The health officer reports that, of twenty-six cases, nine obtained their infection while out of the city, five were infected by contact with typhoid carriers, one was re-allocated to Detroit from a neighboring township, and in eleven instances the source of infection could not be ascertained.

The six cities (Table 5) in the East South Central group (population 1,364,025) again show an increase in the death rate (2.42 in

1939, 2.36 in 1938, 2.10 in 1937). The rate remains far below that of 1936 (3.35) and that of the quinquennial period 1931–1935 (4.81). The actual number of deaths in these six cities increased but one, from thirty-two in 1938 to thirty-three in 1939. For the first time, one of these cities (Chattanooga) is included in the list of those cities

TABLE 5

Death Rates of Six Cities in East South Central States from Typhoid per Hundred Thousand of Population

	1939	1938	1937	1931–1935	1926–1930	1921–1925	1916–1920	1911–1915	1906–1910
Chattanooga...	0.0	1.7	1.7	4.7	8.0	18.6	2.72	3.58 ⋇	—
Luoisville......	0.8*	0.8†	0.6*	2.3	3.7	4.9	9.7	19.7	52.7
Birmingham...	1.0†	1.7†	1.4	3.9	8.0	10.8	31.5	41.3	41.7
Nashville......	3.0†	4.9†	1.2*	5.6	18.2	17.8	20.7	40.2	61.2
Knoxville......	3.8	3.9	3.2	5.7	10.7	20.8	25.3 ⋇	—	—
Memphis.......	5.7†	3.1†	4.9†	7.9	9.3	18.9	27.7	42.5	35.3

*† See footnote Table 1.

⋇ Incomplete data.

TABLE 6

Death Rates of Nine Cities in West North Central States from Typhoid per Hundred Thousand of Population

	1939	1938	1937	1931–1935	1926–1930	1921–1925	1916–1920	1911–1915	1906–1910
Wichita...........	0.0	0.0	0.0	0.7	1.2	6.3	—	—	—
Kansas City, Kan.	0.0	0.0	0.8*	1.1	1.7	5.0	9.4	31.1	74.5 ⋇
St. Paul..........	0.0	0.7	0.0	0.7	1.4	3.4	3.1	9.2	12.8
Duluth...........	0.0	1.0	0.0	1.0	1.1	1.7	4.4	19.8	45.5
Minneapolis.......	0.2	0.0	0.2*	0.8	0.8	1.9	5.0	10.6	32.1
Omaha............	0.4	0.0	1.3	0.9	1.3	3.3	5.7	14.9	40.7
St. Louis..........	0.7†	0.5	1.1†	1.6	2.1	3.9	6.5	12.1	14.7
Kansas City, Mo..	0.9	0.7†	1.4†	1.5	2.8	5.7	10.6	16.2	35.6
Des Moines.......	1.3	0.7	0.7	2.1	2.4	2.2	6.4	15.9	23.7

*† See footnote Table 1.

⋇ Incomplete data.

without a death in 1939. In addition to this there are three cities (Birmingham, Nashville, Memphis) in which one third or more of the reported deaths were stated to be among nonresidents. It is here evident that the surrounding rural areas continue to contribute to the hospital load and thus to the death rate of these urban centers.

Birmingham, with three deaths, reports one among nonresidents; Nashville, with five deaths, records four in nonresidents; Memphis, with seventeen deaths, reports twelve in nonresidents. The health officer of Memphis reports twenty-eight cases in residents: six among white persons, twenty-two among Negroes. The attack rate for Negroes is nearly six times that of the white population. Since Memphis serves as a hospital center for the surrounding territory, thirty-six additional cases were brought into the city for treatment (thirteen white with five deaths, twenty-three Negroes with seven deaths). As one third of the nonresident patients died, it is fair to conclude that only the unfavorable cases were hospitalized, the milder cases not being sent to Memphis. Knoxville records five

TABLE 7

Death Rates of Eight Cities in West South Central States from Typhoid per Hundred Thousand of Population

	1939	1938	1937	1931–1935	1926–1930	1921–1925	1916–1920	1911–1915	1906–1910
Fort Worth	0.5*	2.7†	2.2	4.6	5.9	6.1	16.3※	11.9	27.8
Tulsa	1.3	0.6	0.0	1.1	8.3	16.2※	—	—	—
Houston	1.6†	3.1	2.0	3.2	4.8	7.6	14.2	38.1	49.5※
Dallas	1.6*	4.3†	3.0*	5.4	7.3	11.2	17.2	—	—
San Antonio	2.3*	2.3†	3.8	4.2	4.6	9.3	23.3	29.5	35.9
Oklahoma City	2.7	1.3	3.1	4.3	7.4※	—	—	—	—
El Paso	5.6*	5.9	0.0	4.9	9.1	10.8	30.7	42.8	—
New Orleans	6.9†	5.5†	2.3†	9.6	9.9	11.6	17.5	20.9	35.6

*† See footnote Table 1.

※ Incomplete data.

deaths, four among residents. Louisville deserves special mention as all three deaths were among nonresidents.

The West North Central group (Table 6) (population 2,809,679) shows an increase in the death rate (0.50 in 1939, 0.40 in 1938). In second place in 1938, this group in 1939 just edges out the Mountain and Pacific group for fourth place. The increase in the number of deaths is but three (from eleven in 1938 to fourteen in 1939). As in 1938 there are four cities (Wichita, Kansas City, Kan., St. Paul, Duluth) which report no death in 1939. They are not, however, altogether the same cities. Minneapolis and Omaha, each with one death among residents, have been dropped from the honor list. Minneapolis had reported no death among residents for three

years prior to 1939. Duluth returns to the honor roll after an absence of one year (no deaths in 1936 and 1937). Only one death has occurred in Duluth during the past four years. In 1938 this was the only group in which there was no city with a death rate in excess of 1.0. In 1939 there is no such group, although the West North

TABLE 8

Death Rates of Eleven Cities in Mountain and Pacific States from Typhoid per Hundred Thousand of Population

	1939	1938	1937	1931–1935	1926–1930	1921–1925	1916–1920	1911–1915	1906–1910
Salt Lake City	0.0	0.0	0.7	1.0	1.9	6.0	9.3	13.2	41.1
San Francisco	0.0	0.7	0.6	0.8	2.0	2.8	4.6	13.6	26.3
Seattle	0.3	0.0	0.0	0.6	2.2	2.6	2.9	5.7	25.2
Portland	0.3	0.0	0.3	0.8	2.3	3.5	4.5	10.8	23.2
Oakland	0.3	0.6†	0.3	1.0	1.2	2.0	3.8	8.7	21.5
Los Angeles	0.4	0.6†	0.7†	0.8	1.5	3.0	3.6	10.7	19.0
Long Beach	0.6	0.6	0.0	0.2	1.1	2.1 ※	—	—	—
Tacoma	0.9	0.0	0.0	0.7	1.8	3.7	2.9	10.4	19.0
San Diego	1.0*	0.0	1.6†	1.3	1.0	1.6	7.9	17.0	10.8
Denver	1.9	1.0	2.7	1.8	2.6	5.1	5.8	12.0	37.5
Spokane	2.4†	2.4†	0.0	1.4	2.2	4.4	4.9	17.1	50.3

*† See footnote Table 1.

※ Incomplete data.

TABLE 9

Fourteen Cities with No Typhoid Death in 1938 and 1939

Bridgeport*	Kansas City, Kan.	Salt Lake City
Cambridge	Lowell	South Bend¶
Elizabeth	Lynn†	Utica¶
Fall River†	Milwaukee†	Wichita†
Fort Wayne**	New Bedford†	

* No typhoid death in six years.

** No typhoid death in five years.

¶ No typhoid death in four years.

† No typhoid death in three years.

Central cities maintain second place (Des Moines with 1.3) while the New England cities take first place (New Haven with 1.2). Kansas City, Kan., reports no death among residents for the past three years, while Wichita records no death either among residents or nonresidents for three years. Of six deaths in St. Louis and four in Kansas City, Mo., three each were among residents.

TABLE 10

Death Rates from Typhoid in 1939

Honor Roll: No Typhoid Deaths (Thirty-Four Cities)

Akron	Fort Wayne	South Bend
Albany	Hartford	Springfield
Bridgeport	Jersey City	St. Paul
Cambridge	Kansas City, Kan.	Syracuse
Chattanooga	Lowell	Trenton
Dayton	Lynn	Utica
Duluth	Milwaukee	Waterbury
Elizabeth	New Bedford	Wichita
Evansville	Paterson	Wilmington
Fall River	Salt Lake City	Worcester
Flint	San Francisco	Yonkers
	Somerville	

First Rank: from 0.1 to 1.9 Deaths per Hundred Thousand (Forty-Nine Cities)

Buffalo	0.2	Grand Rapids	0.6	Cincinnati	1.0†
Minneapolis	0.2	Long Beach	0.6	San Diego	1.0*
Chicago	0.3	Philadelphia	0.6†	Tampa	1.0
New York	0.3	Rochester	0.6†	Youngstown	1.1
Oakland	0.3	Washington	0.6	New Haven	1.2†
Portland	0.3	Baltimore	0.7†	Des Moines	1.3
Seattle	0.3	Scranton	0.7*	Tulsa	1.3
Toledo	0.3	St. Louis	0.7†	Miami	1.4
Boston	0.4†	Louisville	0.8*	Dallas	1.6*
Los Angeles	0.4	Norfolk	0.8*	Erie	1.6
Omaha	0.4	Canton	0.9*	Houston	1.6†
Pittsburgh	0.4†	Columbus	0.9†	Peoria	1.6
Providence	0.4	Kansas City, Mo.	0.9	Richmond	1.6
Cleveland	0.5†	Newark	0.9	Denver	1.9
Detroit	0.5	Reading	0.9	Indianapolis	1.9†
Fort Worth	0.5*	Tacoma	0.9	Jacksonville	1.9
		Birmingham	1.0†		

Second Rank: from 2.0 to 4.9 (Seven Cities)

Atlanta	2.2	Spokane	2.4†	Nashville	3.0†
San Antonio	2.3*	Camden	2.5*	Knoxville	3.8
		Oklahoma City	2.7		

Third Rank: from 5.0 to 6.9 (Three Cities)

El Paso	5.6*	Memphis	5.7†	New Orleans	6.9†

*† See footnote Table 1.

The eight cities of the West South Central group (Table 7) (population 2,138,496) report a reduction in the death rate (3.52 in 1938, 3.19 in 1939). The actual number of deaths decreased from seventy-four to sixty-eight. Special mention should be made of El Paso and San Antonio, each reporting six deaths, all among nonresidents; Dallas recording five deaths and Fort Worth one death, all among nonresidents. New Orleans reports thirty-six deaths, twenty-four among nonresidents. Excluding New Orleans, there were thirty-two deaths reported by the cities in this group, of which twenty-one,

TABLE 11

Number of Cities with Various Typhoid Death Rates

	NO. OF CITIES	10.0 AND OVER	5.0 TO 9.9	2.0 TO 4.9	1.0 TO 1.9	0.1 TO 0.9	0.0
1906–1910	77	75	2	0	0	0	0
1911–1915	79	58	19	2	0	0	0
1916–1920	84	22	32	30	0	0	0
1921–1925	89	12	17	48	12	0	0
1926–1930	92	3	10	30	37	12	0
1931–1935	93	0	6	17	28	42	0
1930	93	2	6	30	23	22	10
1931	93	2	6	23	28	22	12
1932	93	1	7	13	29	29	14
1933	93	0	7	18	19	33	16
1934	93	0	9	11	27	23	23
1935	93	0	7	15	18	29	24
1936	93	0	3	15	21	36	18
1937	93	0	1	13	26	26	27
1938	93	0	3	13	14	34	29
1939	93	0	3	7	17	32	34

or two thirds, were among nonresidents. Houston reports six deaths, three among residents; Oklahoma City six, all among residents.

The cities in the Mountain and Pacific states (Table 8) report the same number of deaths as in 1938 (twenty-two) and consequently but a slight reduction in the rate (0.51 in 1939, 0.52 in 1938). While in 1938 there were five cities with no death, in 1939 there were but two (Salt Lake City, San Francisco) such cities, and only Salt Lake City appears on the honor roll for the two years. San Diego with no death in 1938 and two deaths among nonresidents in 1939 deserves special mention. Tacoma, Portland and Seattle, each with one

TABLE 12

*Total Typhoid Rate for Seventy-Eight Cities 1910–1939**

	POPULATION	TYPHOID DEATHS	TYPHOID DEATH RATE PER 100,000
1910	22,573,435	4,637	20.54
1911	23,211,341	3,950	17.02
1912	23,835,399	3,132	13.14
1913	24,457,989	3,285	13.43
1914	25,091,112	2,781	11.08
1915	25,713,346	2,434	9.47
1916	26,257,550	2,191	8.34
1917	26,865,408	2,016	7.50
1918	27,086,696†	1,824†	6.73
1919	27,735,083†	1,151†	4.15
1920	28,244,878	1,088	3.85
1921	28,859,062	1,141	3.95
1922	29,473,246	963	3.26
1923	30,087,430	950	3.16
1924	30,701,614	943	3.07
1925	31,315,598	1,079	3.44
1926	31,929,782	907	2.84
1927	32,543,966	648	1.99
1928	33,158,150	628	1.89
1929	33,772,334	537	1.59
1930	34,386,717	554	1.61
1931	35,137,915	563	1.60
1932	35,691,815	442	1.24
1933	35,691,815	423	1.18
1934	35,401,715	413	1.17
1935	35,401,715	348	0.98‡
1936	36,216,404	336	0.93§
1937	36,771,787	280	0.76 ✱
1938	36,972,985	248	0.67 ✱ ✱
1939	37,112,665	239	0.65 ✱ ✱ ✱

* The following fifteen cities are omitted from this table because data for the full period are not available: Canton, Chattanooga, Dallas, Fort Wayne, Jacksonville, Knoxville, Long Beach, Miami, Oklahoma City, South Bend, Tampa, Tulsa, Utica, Wichita, Wilmington.

† Data for Fort Worth lacking.

‡ The rate for ninety-three cities in 1935 was 1.03 (total population 37,437,812, typhoid deaths 385), whereas in 1930 it was 1.64 and in 1933 and 1934 it was 1.24 and 1.25 respectively. The 1931–1935 average for the ninety-three cities is 1.31.

§ Rate for ninety-three cities in 1936 was 0.96 (total population 38,249,094, typhoid deaths 365).

✱ Rate for ninety-three cities in 1937 was 0.82 (total population 38,885,435, typhoid deaths 318).

✱ ✱ Rate for ninety-three cities in 1938 was 0.74 (total population 39,143,556, typhoid deaths 291).

✱ ✱ ✱ Rate for ninety-three cities in 1939 was 0.67 (total population 39,354,549, typhoid deaths 265).

Special Note:—Deaths for 1936 have been corrected, as Yonkers originally reported seven deaths and later corrected report to one death.

resident death, have been dropped from the honor roll. Spokane records three deaths, two among residents. Denver reports six deaths among residents and does not keep a record of deaths among nonresidents. Los Angeles records six deaths, five among residents. In October, 1939, thirty-five cases in the city and ten additional cases in the county were caused by a carrier who contaminated chocolate éclairs.

The number of cities with no death from typhoid has increased to thirty-four. In 1936 there were but eighteen such cities, in 1937, twenty-seven, in 1938, twenty-nine. Of particular significance are the data in Table 9, which furnishes the names of fourteen cities

TABLE 13

Total Typhoid Death Rate per Hundred Thousand of Population for Ninety-Three Cities According to Geographic Divisions

	POPULATION	TYPHOID DEATHS		TYPHOID DEATH RATES				
		1939	1938	1939	1938	1937	1931-1935	1926-1930
New England	2,657,824	6	12	0.23	0.45	0.45	0.70	1.31
Middle Atlantic	13,602,500	51	59	0.37	0.44	0.51	0.80	1.40
South Atlantic	2,622,237	27	46	1.03	1.74	1.96	2.70	4.50
East North Central	9,883,376	44	35	0.44	0.35	0.62	0.75	1.29†
East South Central	1,364,025	33	32	2.42	2.36	2.10	4.81	8.31
West North Central	2,809,679	14	11	0.50	0.40	0.76	1.24	1.83
West South Central	2,138,496	68	74	3.19	3.52	2.34	5.36	7.32‡
Mountain and Pacific	4,276,412	22	22	0.51	0.52	0.68	0.88	1.80

† Data for South Bend for 1925–1929 are not available.
‡ Lacks data for Oklahoma City in 1926.

with no typhoid death in 1938 and 1939. Several cities have continued their excellent records. We again find (as in 1938) three cities in the third rank (Table 10). It should be emphasized, however, that in El Paso all deaths were stated to be in nonresidents and in Memphis twelve of seventeen deaths were in nonresidents. As repeatedly stated, several other cities in the first and second rank would appear in the honor roll were they not charged with deaths in nonresidents. Table 11 continues to show a definite swing "to the right" with a marked reduction in the number of cities with a rate of 2.0 and above.

For the seventy-eight cities (Table 12) for which data are available

since 1910, there occurred 239 deaths from typhoid in 1939, which is the lowest of record (248 in 1938, 280 in 1937). The rate for this group of cities is for the fifth consecutive year less than 1.0. The rate for the ninety-three cities studied in 1939 is also below 1.0 (0.67) and below the corresponding rate for 1937 (0.74). This annual review again shows a downward trend in the death rate from typhoid in the large cities of the United States. No outbreak of epidemic proportion has been recorded in these cities. Routine vaccination of the population is not practiced except under flood conditions. In progressive communities vaccination is urged for contacts to cases and for persons who travel widely in countries where sanitary conditions are not of the best. Noteworthy gains have been made throughout the country. The New England and Middle Atlantic groups have maintained their excellent rates of many years' standing.

ABSTRACTS OF WATER WORKS LITERATURE

Key. **31:** 481 (Mar. '39) indicates volume 31, page 481, issue dated March 1939. If the publication is paged by issues, **31:** 3: 481 (Mar. '39) indicates volume 31, number 3, page 481. Material enclosed in starred brackets, ★[ ]★, is comment or opinion of abstractor. Initials following an abstract indicate reproduction, by permission, from periodicals as follows: *B. H.—Bulletin of Hygiene (British); C. A.—Chemical Abstracts; P. H. E. A.—Public Health Engineering Abstracts; W. P. R.—Water Pollution Research (British); I. M. —Institute of Metals (British).*

WATER SUPPLY—GENERAL

Water Supply and Sewerage Improvements Constructed During 1939. R. H. Markwith. Annual Report of Div. of San. Eng., Ohio Dept. of Health, 1939, p. 5. Includes appendix of major water supply, sewerage and industrial waste improvements completed or placed under construction during year. Water supply improvements undertaken in 24 cities, 44 villages and 5 counties. Major ones are new Nottingham filtration plant in Cleveland and new Lake Erie supplies, including filtration plants and pumping stations, for Sandusky and Toledo. In villages, 20 new softening plants completed, 1 under construction; 15 new water works systems installed, 3 under construction; and, in addition, several new purification, iron-removal and conditioning plants, and extensions to regular equipment, were projected. Estimated cost of these improvements total $9,304,900, which, together with $9,136,575 for new sewerage projects, and $117,000 for school-building water and sewerage projects, represents greatest effort in construction in history of dept.—*Ed.*

Design Features of the Toledo Water Supply Project. Paul Hansen. Ohio Conf. Water Purification, 19th Ann. Rept. 26 ('39). Toledo has decided to abandon its Maumee R. supply in favor of one from Lake Erie which will be softer (125 p.p.m.) and of better and more uniform quality. Structures not readily enlarged were designed for requirements of yr. 2000 and remainder for those of yr. 1970. Intake, extending 2 mi. from shore into 20′ of water, terminates in exposed crib, considered necessary to combat ice, which may occur in 2 forms—frazil and pressure. Intake 8 mi. long would be required to reach into 30′ of water. Inlet velocity will not exceed 0.25 ft. per sec. at flow of 180 m.g.d. (estimated max. in yr. 2000). Intake ports are shaped to facilitate dislodging frazil ice and bringing it into the interior. Superstructure provides accommodation for 4 men for several months. Precautions to prevent contamination at intake will be taken. No laundering to be permitted, liquid wastes will be conveyed by boats to safe distance for disposal, and solid wastes incinerated. Intake conduit is 108″ in diam. and 15,490′ long. Lake section

will be of reinforced-concrete pipe laid in excavated trench and land section in earth tunnel. Low lift or Lake Erie Pumping Station will contain 3 pumps of 50 m.g.d. and 1 of 55 m.g.d. capacity. All will be horizontal double-suction centrifugal pumps driven by slip ring motors, provision being made for 30% speed variation in 10-11 equal steps. Standby power will be provided by 2 generator units driven by internal combustion engines. Pipe line to filter plant will be about 45,000′ long and 78″ in diam. To control surge, pumps will be equipped with cone type valves and surge suppressors will be used to admit water to line quickly during direct surge. Filter plant has nominal capacity of 80 m.g.d. and consist of 4 basins, providing reaction period of 45 min. and settling period of 3 hr., and 20 filter units, each divided into 2 filters with separate effluent and wash water control, operated at 2 g.p.m. per sq. ft. Reaction chambers are equipped with flocculators of paddle-wheel type. Partial softening is provided for, carbonation basins having retention period of 12 min. being placed at ends of sedimentation basins. Backwash rates up to 25 and surface wash at rates up to 10 g.p.m. per sq. ft. can be employed. Surface wash water will be applied through vertical pipes, spaced on 2′ 9″ centers, fitted with caps with five ⅜″ holes, one vertical and others at about 38° with horizontal. "Wheeler" bottoms will be employed. Filtered water is divided into 2 chambers of 2 and 33 mil. gal. capacity, respectively. Ordinarily, filtered water will discharge into small basin, which will have overflow weir into larger basin. Pumps will draw water from small chamber and thus will have positive head on suctions at most times. Flap-gates will open automatically and admit water from larger basin if pumping rate exceeds filtering rate. Design of plant was largely controlled by fact that all available sites were underlain with thick layer of plastic clay. Large elevated tank of about 1.5 mil. gal. capacity is necessary to prevent sudden reduction of pressure in distr. system during electric power outages. Tank will be used for wash water storage also. Main pumping station will be equipped with 5 pumps, 4 driven by 2-speed synchronous motors and other by single-speed synchronous motor. Capacity of units will vary from 48 to 65 m.g.d. Power will be obtained from 3 independent sources. Trunk main will decrease in size from 72″ at pumping station to 42″ and will be constructed of tar-enamelled steel pipe. Maumee R. crossing is in rock tunnel, 110′ below surface. Completion is expected by Dec. 31, '40.—*R. E. Thompson*.

Construction Features of the Toledo Lake Water Supply Project. O. E. CLARK. Ohio Conf. Water Purification, 19th Ann. Rept., p. 36 ('39). Early history of Toledo's water supply, construction organization for present project, and some construction details are described. Most of work under way consists of pipe laying contracts. All mains are of steel, coated inside and outside with bituminous enamel and jointed with "Dresser" couplings. Steel pipe with "Dresser" couplings will be laid in rock tunnel under Maumee R. and space between pipe and tunnel filled with concrete. Because of low bearing value of soil, harnesses across couplings were employed at pipe ends and angles to distribute load caused by internal water pressure. Harness consists of 2 bolts, one on each side of horizontal center line, threaded through fabricated lugs welded to pipe sides. Flexibility is maintained in this way and by using spherical shaped washer and bearing plate which forms universal

joint just ahead of nut at each end of bolt. On 72" diam. high-service pipes, harness bolts are 4½" in diam. Work is also under way on intake crib. To give protection from waves, contractor sunk a 400' lake freighter filled with sand near intake site. Freighter also provides haven for workmen and storage place for steel piling with which coffer dam is being constructed. It will be removed on completion of work.—*R. E. Thompson.*

Intake Crib and Other Engineering Features of the New Lake Water Supply Project at Sandusky. O. F. Schoepfle. Ohio Conf. Water Purification, 19th Ann. Rept., p. 41 ('39). Existing water supply system, drawing water from Sandusky Bay, was built in 1875 and added to at intervals since that time, filter plant being constructed and enlarged in '09 and '13, respectively, and softening facilities added in '30. Interruptions due to bay level being lowered by high winds and major taste and odor difficulties due to industrial wastes have led to construction of new supply system drawing its water from Lake Erie, on completion of which old plant will be abandoned. New intake consists of 42" welded steel pipe, coated inside and outside with coal tar pitch, terminating 3000' from shore in intake drum housed in white oak crib. "Dresser" couplings were used throughout, ordinary type on 40' lengths of shore section and anchored type on 120' lengths of lake section. Clay bottom of lake was excavated to rock around intake drum and backfilled with large rocks laid on 2½:1 slope to permit movement of ice over structure, 5' above normal lake bottom, with min. of stress. Port velocity at ultimate capacity will be 0.1 ft. per sec. Pipe is laid on upgrade from lake end, where its center line is 31' below mean lake level, to plant end, where center line is 12' below. Pipe turns down 9' in duplicate cells of suction well, making siphon of line, thus increasing capacity. Screening facilities are provided at suction well to remove trash that passes racks at lake end. In emergency, water may be drawn from rock-filled crib at shore line. As site is approx. at mean lake level, plant was built above normal ground level and is protected by earth embankments. Low-lift pumping equipment consists of 4 centrifugal units of 3, 4.5, 6 and 7 m.g.d. capacity, all driven by induction motors. High service is supplied by 3, 5, 5, 6.5 and 8 m.g.d. centrifugal pumps, first 2 driven by induction motors and last 2 by synchronous motors. Second 5-m.g.d. unit is mounted between synchronous motor and 8-cylinder gasoline engine on common bedplate. In event of power outage, gas engine will drive pump and also synchronous motor on same shaft, making latter a generator to provide sufficient current to operate low service pump, miscellaneous small motors and lighting system. All pumps are equipped with cone check valves. Reaction chambers are equipped with vertical shaft paddles and provide 50 min. detention; sedimentation basins are of around-the-end type, providing 4 hrs. detention. The 6 filters, of 1.5 m.g.d. capacity each at 125 m.g.a.d., are equipped with "Wheeler" bottoms. Provision has been made for installation of surface washing system. In lab., booth with fluorescent lighting provides light of constant intensity, approaching that of northern sky, for colorimetric work. New supply will have hardness of 110 p.p.m., compared with Sandusky Bay hardness peaks of 300–350 p.p.m. Cost is estimated at $831,200. —*R. E. Thompson.*

Great Lakes Water for Grand Rapids. ANON. Eng. News-Rec. **124:** 448 (Mar. 28, '40). Details are given of new water supply for city of Grand Rapids, Mich., pop. 170,000. Hardness of present supply from Grand R. ranges from 100 to 300 p.p.m., varying proportion of which is non-carbonate, while lake supply has ave. hardness of 125 p.p.m., mostly in carbonate form. New system will consist of submerged intake, intake pipe, circular pumping station structure and rectangular superstructure, 46" concrete pipeline 30 mi. long to existing filter plant, booster station on 46" line, one 8-mil. gal. and two 16-mil. gal. reinforced concrete res., 2 river crossings, and twelve 46" pipelines in city between distr. system and new res. Construction work on intake line, Lake Shore pumping station, pipeline, and res. described. Intake pipe, which will be 6,000' long, is being constructed of 54" welded-steel pipe coated with bitumatic and laid in 10' trench. It will terminate in conventional timber crib, through outer openings of which velocity will be 0.3' per sec. at 60-m.g.d. rated capac. Back-flushing is provided for. Circular well pit, which extends 31' below lake level in sandy terrain, was dewatered with well points, as was also pipeline trench. Pumps will be below lake level with vertical shafts to motors above lake level. Provision of new res. permitted pipeline diam. to be reduced from 60" to 46".—*R. E. Thompson.*

Water Supply System of Kansas City, Mo. T. D. SAMUEL, JR. W. W. Eng. **93:** 222 (Feb. 28, '40). Water for Kansas City, Mo. (pop. supplied 500,000) taken from Missouri R. at point on opposite bank about 4 mi. north of City. Water treated with lime, alum, ammonium sulfate and chlorine. Intake has capac. of 150 m.g.d.; low-lift pumps include four 35 m.g.d. and one 20 m.g.d. units pumping against 51' head, max. Purification consists of 5 steps: (1) 4 circular pre-sedimentation clarifiers with 4 hr. retention at 100 m.g.d. reducing turbidity from up to 24,000 to yearly ave. of 497 p.p.m.; (2) 30-min. mixing in 2 circular, concrete, 90' diam. tanks with tangential flow where lime and alum is added; (3) prechlorination; (4) settling in 3 coag. basins, capac. 37½ mil. gal. providing 9 hr. settling at 100 m.g.d.; (5) water with ave. turb. of 62 p.p.m. then passes to 3.5 hr. final settling in 2 additional (14½ mil. gal. capac.) basins, leaving with ave. turb. of 22 to 24. Pump station at filter plant equipped with four 35 m.g.d. pumps for secondary pumping, head 48'. There are 2 high-lift pumping stations known as East Bottoms and Turkey Creek Stations; first is electric operated, has two 24 m.g.d. and one 12 m.g.d. units; second steam, has one 20 m.g.d., two 15 and 20 m.g.d., two 20 m.g.d. of other types, and one 30 m.g.d. turbine-driven centrifugal pumps. Total capac. of station 125 m.g.d. Ave. daily consumption 57.19 m.g.d.; per capita use 114.4 gal.; system has 866 mi. of mains with pressure range of 30 to 150 lb., 89,207 taps and 82,001 meters. Cost of operation, maintenance and bond interest $106.82 per mil. gal., revenue per mil. gal. $107.69.—*Martin E. Flentje.*

The Waterworks System of London, Ont. ANON. Eng. Cont. Rec. **53:** 13: 25 (Mar. 27, '40). Public water system was installed in London 62 yr. ago, when pop. was less than 20,000, or ¼ of that now served. To date, over $2,750,000 has been invested. Original supply, from springs at Springbank, 4 mi. west of city on bank of Thames R., producing about 3 m.g.d., is

still operating satisfactorily. Since that time 4 wells have been constructed at Ridout and Horton Sts., producing about 1 m.g.d., 3 located 2 mi. south of that field producing 0.75 m.g.d., 6 in east end of city producing 1 m.g.d., and 10 within 1 mi. of northern boundary producing 1 m.g.d. All wells are in gravel, depth varying from 80′ to 130′. All are now equipped with deep well turbine pumps. Screens in earlier wells gave difficulty due to encrustation, which had to be removed with muriatic acid. Recently constructed wells are of gravel-wall type, equipped with patented shutter screens. Water from Ridout St. wells is aerated, settled, and passed through battery of 6 pressure filters to remove hydrogen sulfide and iron. Ultimately, a treatment plant, possibly including softening, will be constructed in north end of city. Consumption averages 71.3 gal. per capita per day. Every consumer is metered and about 84% of pumpage can be accounted for. Supply is of excellent quality but is chlorinated as precautionary measure. Hardness is 18-20 g.p.g. (Imp.). Water works is self-supporting. Fire protection charges average $18 per hydrant, ave. domestic bill is $12.42 per yr., and largest consumers obtain water at less than 7¢ per 100 cu. ft.—*R. E. Thompson.*

Seventy-Third Annual Report of the City of Erie Bureau of Water, Year Ending Dec.31, 1939. 74 pp. Detailed financial and operating statistics are given. Daily consumption by estimated population of 121,000 averaged 24.12 mil. gal., equivalent to per capita consumption of 199.35 gal. Cost of collecting, purifying and pumping water was $33.56 per mil. gal. No. of gal. of water pumped per lb. of coal consumed averaged 384.55. Expenditures, exclusive of depreciation, exceeded income by $12,353.97. Value of water furnished without charge for municipal purposes totaled $63,936.66. There are now 32,671 services of which 6,024 are not in use. In '39, 257 services were added at ave. cost of $23.63. Ave. amounts of chemicals used at Chestnut St. and West Filtration Plants, respectively, were: alum 0.237 and 0.234 g.p.g., chlorine 1.53 and 1.57 lb. per mil. gal., activated carbon 2.79 and 2.84 lb. per mil. gal. Percentage of wash water used at the 2 plants was 2.13 and 1.29, respectively. Tabulation is included showing no. of bacteria, coliform bacteria, turbidity, alkalinity and color of raw and filtered water at filtration plants and ave. water temp. Of 912 samples of water examined for coliform bacteria in 10 and 1 ml., not a single positive result was obtained. Description of works and schedule of water rates are appended.—*R. E. Thompson.*

Annual Report of the Bureau of Sanitary Engineering of the Maryland State Department of Health, Year 1939. George L. Hall. 27 pp. Activities of Bureau in fields of water supply, sewage disposal, industrial waste treatment, stream pollution, etc., are reviewed. Total value of work represented in plans submitted for review during yr. amounted to approx. $8,420,000, decided increase over that expended in former yr. Typhoid cases reported totaled 181 and there were 18 deaths, case and death rates being 10 and 1 per 100,000, respectively, lowest in history. Death rate for Baltimore was 0.2 and for State, exclusive of Baltimore, 1.7. Mine sealing operations were continued in 2 counties. Four new water supplies were developed during yr., and there are now 144 public supplies in state. 70 of these receive treatment. Brief details are given of new projects and improvements to existing plants undertaken during yr.—*R. E. Thompson.*

A Billion Gallons Not Enough. CHARLES M. CLARK. Eng. News-Rec. **124:** 513 (Apr. 11, '40). Background of N. Y. C.'s Delaware R. project and its relationship to present sources of supply and threatened water shortage are outlined. Principal sources of supply are impounded surface supplies from Croton and Catskill watersheds, which have estimated safe yield of 300 and 510 m.g.d., respectively; actual delivery during '39 was 295 and 610 m.g.d., respectively. Additional water is obtained from ground sources on Long Island and Staten Island, from which city-owned wells extracted 58 m.g.d. and private water company wells 61 m.g.d. Exhaustion of existing sources by '35 was anticipated in '21 but, as result of the depression and activities in reducing underground and plumbing leakage, consumption has not equalled that of '30 until this yr. During last 4 yr. consumption has equalled or exceeded available supply; in '39, ave. consumption was nearly 40 m.g.d. in excess of total dependable supply of satisfactory water. As result of unusually dry weather in '39, city's res. were only about ⅓ full in Feb. '40. Wells are being restored on Long Island and vigorous campaign of house-to-house inspection is under way. Delaware R. project will eventually bring 540 m.g.d. from tributaries of Delaware and Hudson Rivers. Principal elements are 3 dams and res., interconnected by tunnels, and 85-mi. pressure tunnel, longest ever constructed. Essential contracts for first 100 m.g.d. are well under way, about 47 mi. of tunnel have been excavated, and lining has been begun on about 5 mi. At earliest, no water can flow before '44.—*R. E. Thompson.*

A Water Supply is Born. FARLEY GANNET. Eng. News-Rec. **124:** 577 (Apr. 25, '40). In '04, filter plant was constructed on island in Susquehanna R. opposite downstream section of Harrisburg, Pa. Although protected by dike 3′ higher than highest flood in century, flood of Mar. '36 exceeded elev. of dike by 2′ and put plant out of commission for 10 days. Fortunately, 50-mil. gal. distr. reservoir was full at time and by reducing consumption from 11 to 5 m.g.d., making connections to suburban water systems, and hauling water in gasoline trucks, acute shortage was avoided. As result of this experience, availability of near-by gravity source, and $1,250,000 P.W.A. grant, new supply system, which will produce 50% in excess of present needs, is under construction. New system consists of 100′ earthfill dam, storing 7 billion gal., on Clark's Creek 20 mi. northeast of city, and 42″ reinforced-concrete gravity conduit, 22 mi. long, to existing distr. res. Watershed is practically uninhabited and filtration will not be required. Saving effected in pumping and filtering costs is more than sufficient to pay interest and amortize bonds for city's share of cost ($1,600,000). Chlorination plant will be located at dam site. Clark's Creek water is much softer than Susquehanna R. water and it is estimated that citizens will save nearly $100,000 per yr. on soap bills.—*R. E. Thompson.*

We Built a Modern Water Works Plant in Keokuk (Iowa). WALTER L. GARRISON. Pub. Wks. **71:** 1:15 (Jan. '40). Since acquisition from private company, Keokuk, Iowa water plant and system enlarged by 10 mi. of 6″ to 16″ pipe, (new total 37 mi.), 90 fire hydrants (new total 281), new softening and purification plant and new 1 mil. gal. elevated tank. Water taken from Miss.

River; 3 pumps two 3 m.g.d. and one 1½ m.g.d. capac. Treatment includes softening, recarbonation, filtration and activated carbon treatment; alum, iron sulfate, carbon, lime and ammonium sulfate chemicals used. Settling basin has capac. of ½ mil. gal., divided into 2 sections, covered and equipped with sludge removal system. This 3 m.g.d. capac. plant has 4 filters, clear well of 157,000 gal., and supplies ave. of slightly over 1 m.g.d. in cold weather and 1½ m.g.d. in warm. System 98% metered.—*Martin E. Flentje.*

Water Supply and Sewage Systems for Heron Bay Townsite. A. W. Lash. Eng. Cont. Rec. **53:** 1: 7 (Jan. 3, '40). Village of Heron Bay South, District of Thunder Bay, Ont., located on Pic R., was established by Ontario Paper Co., in conformity with its policy of providing comfortable living quarters for its staff. Water supply system consists of intake well on Pic R., 20-g.p.m. pump which can be driven either by elec. motor or gasoline engine, automatic hypochlorinator, alum pot, 500-gal. settling tank, 2 vertical pressure filters that have a combined capacity of 20 g.p.m. at filtering rate of 2.0 gal. per sq. ft. per min., and 1500-gal. steel pressure tank, air supply being provided by 2.73-cu. ft. per min. air compressor. Operation of pump is automatically controlled to maintain pressure between 60 and 80 lb. per sq. in. Intake well consists of 30″ precast concrete pipes placed vertically and connected with river by 8″ steel pipe. Hypochlorite is introduced at pump suction. Alum is required only when water is muddy following freshets and heavy rain storms. At such times filtration rate can be reduced by bypassing portion of water, when demand permits it. Distribution is through hard-drawn streamline copper pipe. Water for fire protection is delivered through separate 6″ cast iron mains supplied by 500-g.p.m. pump that takes its suction from same well as domestic pump. Interruption in power supply from Black R. Falls is guarded against by gasoline-engine-driven generator set installed at town site. Regular analyses of raw and filtered water have shown very low bacterial counts, watershed being practically uninhabited. Sewage disposal is provided for in septic tank and tile drains to gravel percolation bed.—*R. E. Thompson.*

A New Water System (Narragansett, R. I.). Fred J. Coggeshall. Am. City, **55:** 5: 37 (May '40). With new water system put into operation in summer of '39, it is believed town of Narragansett and adjacent Point Judith area (summer resort) will develop rapidly into more of all-year-round type of community. In addition to convenience of having a water system, fire protection it affords and attendant decrease in cost of insurance will attract active property building. In planning project, town had applied for and been granted Federal aid, but later rejected offer and obtained necessary financing by bond issue. Before bids had been advertised, a hurricane occurred and many residents, particularly fishermen, were deprived of means of livelihood. To meet emergency, commencement of construction was speeded up to provide work for distressed townspeople. Bids were solicited on 3 kinds of pipe—asbestos-cement, steel and cast iron. Asbestos-cement was selected. Pipes were laid deep enough to provide earth cover of 4½′, sufficient in this area to protect against freezing. All dead ends have either a hydrant or a 1-inch blow-off connection to release air or to bleed lines. Consistent with similar

appliances throughout town, all hydrants and valves open to the right. Water is purchased from local commercial water company, supply being metered and credit allowed for all water going back through a reverse-flow meter. System includes about 9 mi. of mains and an elevated tank of 250,000 gal. capacity.—*Arthur P. Miller.*

Improvements in Water Works at Albert Lea, Minn. ANON. Am. City, **54:** 12: 54 (Dec. '39). Constant demands over a period of 6 years by a growing population for increased water supply necessitated and brought about steady improvements not only in the character of the water but in its servicing and distribution. One of the notable improvements is the new 1,000,000-gallon elevated water-storage tank equipped with a device which causes the hydrogen to be deposited on the plates of the tank rather than permitting attacking of the steel by the water, and so obviates deterioration of the tank. Perhaps most interesting of changes made in this plant is the conversion of circular reservoir into a settling basin. Aeration and the application of chlorine and lime has improved the character of the water. Albert Lea's experience shows that in modernizing a plant, old structures need not be discarded but can be usefully retained to advantage. The well water has from 2 to 3 p.p.m. of iron and prior to the improved methods installed, this considerably high iron content was increased by the iron bacteria prevalent in the water. Since modernization, iron in the water which finally reaches the consumer never exceeds 0.5.—*Arthur P. Miller.*

Important Assets of Gloucester (Mass.) Water Supply. ANON. Am. City, **54:** 12: 57 (Dec. '39). A new reservoir hewn in hard granite is being constructed at Gloucester to replace one now in use which is deficient in storage capacity and pressure. Houses situated on higher parts of city were not being adequately supplied. The new 6 mil. gal. covered reservoir will add 12½ lb. pressure throughout the city and in addition will have twice the storage space. Construction was started in January, 1938, and completion is expected about September, 1940. Work was done entirely by relief labor and has provided employment for about 50 men on the average. The reservoir is "divided transversely by a concrete center wall to permit unwatering and cleaning one section without putting the entire reservoir out of service." Construction was planned so that as much as possible of the concrete work could be accomplished before the onset of cold weather, most of the rock excavation being left for winter months. Estimated total cost is $159,790.00, towards which Federal funds contributed will be $95,450.30, the city bearing the remainder.—*Arthur P. Miller.*

Extensions and Improvements to Scarboro (Ontario) Township Waterworks. E. M. PROCTOR. Can. Engr., Water and Sewage. **78:** 6: 16 (June, '40). To provide sufficient capacity to meet terms of contract to supply water to Township of East York, following extensions and improvements are being made to Scarboro water works: 20″ force-main extending through tunnel and shaft from low-level pumping station on shore of Lake Ontario to filter plant on adjoining high land; 2000-g.p.m. low-lift pump; 20″ Venturi meter and recorder; mixing

chamber and flocculation tank with dry-feed equipment; chlorinator and wash water pump. Supply is derived from L. Ontario through 24″ intake, extending 2400′ from shore, with min. capacity of 5 m.g.d. Filter plant has capac. of 2.5 m.g.d. and there is 1.5-mil. gal. filtered water storage. Existing mixing chamber, which is overloaded, will be replaced by Dorr-type flash mixer and flocculation tank. Powdered alum will be employed. Estimated cost of improvements is $70,000.—*R. E. Thompson.*

Past, Present and Future of "East Brabant" Water Company. W. F. J. KRUL. Water (Neth.) **24:** 45 (Mar. '40). Municipal water supplies have grown rapidly; 70% of the population of Netherlands can be connected. First regional system, South Beveland, required water from the mainland, because no water was available on island, making cooperation of various municipalities essential. Same holds for a series of other islands in Zeeland. Rentability was as a unity considered necessary from start in contrast with similar regional supplies in other countries where financing is conducted on a local basis and expenses pooled. Provincial supply in North Holland is based on same premises of rentability-parity. Technical service is decentralized with equal charges for all connections. All 11 provinces have accepted principle of provincial supply instead of state or city supplies. "East Brabant" case is used to show development of concept "rentability-parity." Technical centralization is practiced by center-cities supplying water to surrounding communities; center-cities combine according to geographic location; charges are same for all. Greatest difficulty was to persuade towns with favorable conditions and ample supplies to combine with less favorably situated, small communities. Capitalization was uniform at 4½%. Principal factors involved are: (1) equalization for largest possible region in regard to construction, development and distribution (77 communities for East Brabant); (2) proper, continued and decentralized supply; (3) most economic technical management; (4) province is co-owner; (5) community shares about $50 per 200 connections; (6) construction capital borrowed by combined communities guaranteed by province; (7) all connections obligatory and made within 5 years to assure rentability; (8) details of geological formation of district are given; and (9) all underground supplies.—*Willem Rudolfs.*

The Tilburg Water Plant, S. Brandenburg. Water (Neth.). **23:** 223 (Dec. 29 '39). This private distribution system and treatment plant was placed in operation in 1895 and gradually extended. Stocks offered to the public and city authorities without takers at 90, now sold at 350. Substantial business men and authorities warned against undertaking. Water is obtained from 47 wells, 30 in operation over 40 yr. and still yielding same. Water is furnished to about 100,000 people and 108 industrial establishments, amounting to about 4.75 m.g.d. Treatment consists of aeration and double rapid filtration. Filters have surface area of 2,500 sq. m. After aeration for oxidation water is filtered through 24″ of sand 1–2 mm. in size with a velocity of 2.60 m./hr. After pre-filtration aeration is repeated and water again filtered through 24″ sand of ½–2 mm. size at a rate of 0.38 m./hr. Backwashing is practiced only for pre-filters with velocity of 45 m./hr. For backwashing 75 cu.m.

water is used; filter runs are about 100 hr. with an average of 65 cu. m./hr. filtered. Results are:

	Fe	CO_2
Raw water	3.5	35.5
After aeration + pre-filtering	0.2	15.0
After second aeration	0.2	11.0
2 x aeration and 2 x filtering	0.1	10.0

Plant is still operated privately, but when dividends are more than 5% water must be delivered free to public bldgs. and schools. Water for fire protection is free and a pressure of 25 m. above street level must be maintained (records show 30–38 m.).—*Willem Rudolfs.*

Supply Project of l'Alpe-d'Huez. ANON. L'Eau. (Fr.) **32:** 67 (Jun. '39). (From Le Petit Dauphinois Nov. 6 '38). Huez, 250 pop., has invested 500,000 fr. in water supply and sewerage systems, besides govt. subsidies, to insure its development as a winter sports center. The water is obtained from the foot of the Rousses and siphoned into a 500 m. storage basin. Consumption is over 600 liters per min.—*Selma Gottlieb.*

Supplying Potable Water and Treating Sewage and Rain Water in the City of Jujuy (Argentine Republic). M. F. PETRE. Bol. Obras Sanitarias Nacion (Buenos Aires). **2:** 588 ('38). Until '02, water supplies of Jujuy, were derived from 3 sources; water from river Chico was conveyed to town in aqueducts and was used for agricultural purposes; well water was used for general purposes other than drinking, and potable water was obtained from infiltration pits constructed in banks of rivers Grande and Chico. In '02 an infiltration gallery, yielding 1,500 cm. per day, was constructed under river Reyes. Subsequently 2 more infiltration galleries were added, and 2 distribution reservoirs built. Population of about 14,000 now receives about 500 liters of water per head per day, for all purposes. Sewage works comprises detritus tank, screens, 2 Emscher tanks, 8 percolating filters, 4 with fixed distributors and 4 with rotary distributors, and 4 digestion pits in which sludge from the Emscher tanks is stored before being utilized as fertilizer; final effluent is discharged to river Grande. Rain water is discharged to rivers Grande and Chico from separate drainage system.—*P. H. E. A.*

Water Works Development in Venezuela. H. C. PLUMMER. Pub. Wks. **69:** 7: 25 ('38). Ministry of Public Works in Venezuela recently created Aqueduct Board to deal with problems of water supply. Steep gradients, soil erosion, and severe seasonal variation in rainfall are among the problems encountered. Description is given of res. being constructed to provide for water supply of Caracas, a town of ¼-mil. pop.—*W. P. R.*

Water Supply on Upper Salt River, Arizona. *Discussion of previous paper.* Proc. A. S. C. E. **66:** 553 (Mar. '40). DANA M. WOOD: Author mentions use of helpful tool—ratio to mean flow. Method can be used for extending short record to longer one by comparison with gaging station showing similar dis-

tribution of runoff in shorter period. Can be used, in extreme cases, for determining a possible duration curve by assuming percentage runoff from rainfall records where no runoff records exist, and by assuming its distribution as at some actual gaging station, provided good judgment is used in choosing runoff record that is assumed typical of site under consideration. In part of his analysis author incorrectly assumes same relative rainfall distribution during year, regardless of its type. That this is far from what actually happens scarcely needs proof. *Ibid.* **66**: 1117 (June '40). George F. McEwen: Although tree-ring measurements are indicators of amount of precipitation, various other factors often have great influence, thus obscuring any relation of tree-ring thickness to precipitation. In this arid region, however, enough evidence has been obtained in support of a sufficiently close correlation to make it possible to interpret tree-ring measurements and to determine general trends and cycles in precipitation over hundreds or even thousands of years.—*H. E. Babbitt.*

Water Supply, Sewerage, and Other Public Services in Country Districts of Low Rateable Value. J. R. OXENHAM. (*Papers contributed for the Public Works, Roads and Transport Congress and Exhibition Council, London,* '39). Discusses difficulties involved in provision of water supplies, sewerage, and other public services in rural districts. Still 3,881 parishes in England and Wales not provided with piped water supply. These are mostly remote or situated in flat clay areas with no underground sources of supply; rateable value is generally low and cost of providing piped supply is high. In '34 Govt. provided grant of a million pounds to help finance rural water supply schemes, but aid is no longer available. Several typical schemes, recently carried out, are described. Water was obtained from wells, or by extension of existing mains, or by purchase in bulk from neighboring water company. Cost of providing water supply in rural area is nearly twice that for well-planned town of same population. Costs per gallon of treating and pumping water for small supplies are high in comparison with larger plants. Use of asbestos cement or light spun cast-iron pipes may reduce cost of mains. Provision of stand-pipes or street fountains should be avoided when possible, but often strong demand for them in country districts where villagers are not used to having supplies in their houses. For many isolated dwellings or small villages it is uneconomic to provide piped water supplies, and wells or rain-water tanks must be used; under Public Health Act, '36, it is possible to insist on sanitary construction. Provision of sewerage systems in country districts less advanced than provision of water supplies, but need is increasing as number of piped supplies increases. A few sewerage schemes recently completed are described. Cost in rural areas is over 50% greater than for similar population under planned urban conditions. Although it would appear that disposal on land could be generally adopted in country districts, frequently necessary to construct disposal works. As skilled attention is not available, trickling filters are generally used. Trouble may be caused by strong trade wastes as there is little dilution available compared with that at sewage works in urban districts. Where there is a piped supply and no sewerage system, cesspools are usually installed. Recommended that cesspools be emptied by sanitary authority.—*W. P. R.*

Results of Water Economy in the Industrial District on the Right Bank of the Rhine. A. RAMSHORN. Städtereinigung. **31:** 183 ('39). Author reviews work which has been carried out by Ruhrverband, Lippeverband and Emschergenossenschaft, in improving condition of Ruhr, Lippe, and Emscher rivers, in promoting measures for control of floods, in treating trade waste waters, and in constructing sewage works. These authorities deal with a heavily populated industrial area of Germany where problems arising from subsidence of land due to coal mining are serious. Emschergenossenschaft is carrying out a program for widening Emscher, and has constructed 24 sewage works since its formation in '04; sufficient degree of purification is attained by settling sewage. Sludge is digested and sludge gas utilized. Brief description of sewage works of Alte Emscher and Emscher-fluss works. In Emscher district there are 16 plants for recovery of phenols from gas works waste waters, 4,000 tons of crude phenol being recovered each year; 4 such plants in Ruhr district and 2 in Lippe district. Course of Lippe has been reconstructed by Lippeverband. River water is used chiefly as cooling water; too saline to serve as water supply. Sewage works constructed at Hamm and at Kamen. Ruhrverband deals with problems of sewage disposal and also of water supply; 500 mil. cu.m of water taken from Ruhr each year to serve as water supplies. During last 30 yr., 12 impounding res. have been built on Ruhr, with total capacity of 260 mil. cu.m. Pollution of river by sewage and industrial wastes reduced by diverting sewage, formerly discharged into Ruhr between Oberhausen and Duisburg, into Rhine. Self-purification in river increased by constructing artificial lakes. The Ruhrverband and Ruhrtalsperrenverein are to unite to form a single authority.—*W. P. R.*

Practices from a City Water Works. L. A. SMITH. Eng. News-Rec. **124:** 531 (Apr. 11, '40). Brief outline of 5 practices which have been found useful in Madison, Wis.: (1) Permanent records of supply and distr. system are kept on 18 x 24" cloth-backed paper, showing services, hydrants, valves, and location and size of mains. Photostatic copies are made for use of foremen, service-crews, etc. (2) Bare spots on pavement after light snowfall often disclose location of leaks. (3) Survey was recently made of all services. (4) Daily reports of pumpage and costs are available at 9 A.M. on succeeding day. (5) Men are kept on duty in service bldg. from 5 P.M. to 1 A.M. In addition to answering service calls, these men wash and grease the trucks.—*R. E. Thompson.*

Replacement of and Economy in Raw Materials in Water Works Practice. H. GOTTING. Gas- u. Wasser. **81:** 393 ('38). Author described use of raw materials easily available in Germany for constructional purposes in water works, water distribution systems, and domestic water supply installation. Screens in wells may be made of ceramic materials; mains and pipes of cement or reinforced concrete; thickness of cast iron and steel pipes for distribution of water may be decreased, particularly if they are lined with bitumen as a protection against corrosion; pipes carrying washing water in filters may be of asbestos cement, porcelain, or concrete; and electrical cables of aluminum or its alloys may replace copper cables. Properties and application of Sinterit for pipe points, and of Mipolam, porcelain, glass, and Kuprema for domestic fittings are described.—*W. P. R.*

The Optimum Thickness of Insulation for Canadian Homes. J. D. Babbitt. Eng. J. (Eng. Inst. Canada) **23:** 20 (Jan. '40). Severity of Canadian winters makes it necessary to insulate all dwellings and other structures that require heating. A house cannot be perfectly insulated. Application of insulation follows law of diminishing returns. Reduction in heat loss as insulation thickness is increased is geometric. Reduction in heating costs is arithmetic. Relative costs of fuel and insulation have an important bearing on problem. More use should be made of cheaper one. Insulation in general, including weatherstripping and double windows should be applied first to points of major heat losses. For this reason weatherstripping and double windows take precedence over wall insulation. Different weather conditions exist for different parts of country. Total amount of cold weather controls annual cost of heating. Min. temp. controls size and fuel burning capacity of heating

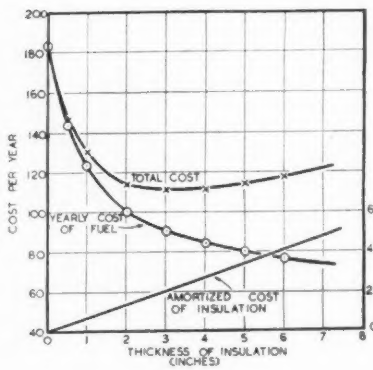

Fig. 1

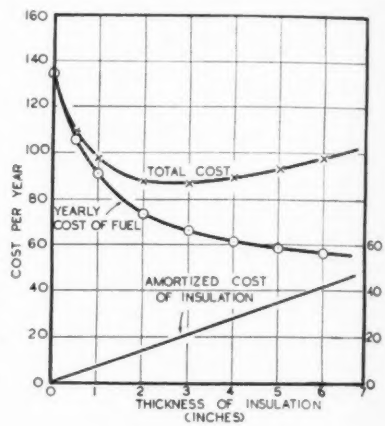

Fig. 2

Fig. 1. Cost Curves for Group 1, 5,500 degree-days
Fig. 2. Cost Curves for Group 2, 7,500 degree-days

plant. Experience and investigations have shown that seasonal cost of heatng varies directly with variation of temp. below 65°F. "Any estimate of amount of insulation to be used in a building must make allowance for weather conditions prevailing where building is to be built. As regards fuel consumption, important feature is not necessarily min. temp. which may be experienced but rather combined effect of low temp. and length of heating season. Min. temp. does have a direct effect on heating arrangements in that heating equipment must be of sufficient size to take care of min. but consumption of fuel depends on length of heating season as well as on temp. difference. In order that both these factors should be included in an estimate of fuel consumption, concept of *degree-days* has been introduced. This, as its name implies, is a unit based upon temp. difference and time. For any one day there exist as many degree-days as there are degrees Fahrenheit difference in temp. between ave. outside air temp., taken over a 24 hr. period, and a temp. of 65°F. Choice of 65°F. as base temp. has been made as result of an investigation carried out by American Gas Assn. which showed that in heating of residences gas con-

sumption varied directly as difference between 65° and outside temp. Has subsequently been found that this also holds good in case of other fuels. A summation of degree-days for each day in year gives total degree-days in year. This forms a basis upon which yearly fuel consumption can be estimated." Table shows degree-days ('31 data) for 9 Canadian cities. These are grouped into 4 classes; Vancouver and Pacific Coast 5,000 to 6,000 degree-days; Halifax and Toronto 7,000 to 8,000 degree-days; Ottawa, Montreal and neighboring districts 8,000 to 9,000 degree-days; prairie cities 10,000 to 11,000 degree-days. Sample calculations of heat losses based on data taken from Guide of A.S.H. & V.E. are made for 5,500, 7,500, 8,500 and 10,500 degree-days. For each condition, curves of amortized cost of insulation, yearly cost of fuel and total cost are plotted (see Figs. 1 and 2). These calculations show that 2½", 3", 3½" and 4" of insulation respectively, are required for the 4 conditions of degree-days. Total cost curve is flat at its low point, showing that within moderately wide ranges of insulation thickness, total cost remains practically the same. Decreasing thickness raises total cost more rapidly than increasing it a corresponding amount. Author closes by reviewing basic assumptions used in calculations and discussing effect of variations from assumptions on solution of problem.—*Homer Rupard.*

Tornado Hazards Removed from a Kansas Water Supply. K. L. BRODE. Am. City, **55:** 1: 38 (Jan. '40). Because the pumping plant of the Newton, Kansas water works was destroyed by a tornado in '23, recent improvements in the system were designed to prevent a recurrence of this along with facilities to provide increased storage for fire protection. For additional storage, a 130-ft. elevated steel tank and 2 surface storage tanks are specified. To protect pumping facilities the booster station is constructed underground.—*Arthur P. Miller.*

Conversion of Buses into Tankers. ANON. Wtr. & Wtr. Eng. (Br.) **42:** 203 (May, '40). Two Regal chassis, purchased from London Transport, have been cleverly converted into special air raid precaution vehicles for use of Sheffield Corp. Water Works. Each completed vehicle is so arranged that in addition to main function of supplying water to works premises and refilling stationary tanks, it can also be made available for consumers wishing to draw off supplies into their own vessels. Conversions are interesting as showing how economies can be obtained by adapting old chassis for new forms of work.—*H. E. Babbitt.*

A New Water-Tower Houses Town Offices. ANON. Am. City **55:** 4: 70 (Apr. '40). Unique experiment in utilizing space successfully demonstrated in Lake, Wisc., by its new Town Building. Concrete hectagonal-shaped structure houses not only million-gallon water tank enclosed in central tower running entire height from ground to spherical top, but also water works pumping station and accessory equipment, all town offices and the town jail. In planning new water system, municipality elected to build an artesian well in preference to using Lake Michigan water. Determined that cost of this well and necessary accompanying softening equipment (analysis of well water showed hardness of 26 g.p.g. and some iron content) compared favorably with

estimated cost of building two-mile tunnel which would have been required to eliminate pollution and stoppage caused by seasonal ice congestion, plus a filtration plant, had lake supply been chosen. In building distribution system, 43 mi. of bell-and-spigot pipe in 18′ lengths were laid. Using longer pipe instead of conventional 12′ length proved doubly profitable minimizing number of joints to decrease possibilities of leaks, and saving $100,000 in cost of laying mains. As for building itself, soundproof walls in pump room, and other comfort-producing features add to attractiveness of interior. $1,300,000 is approx. cost of entire project, including engineering and interest on construction.—*Arthur P. Miller.*

The Combined Light, Water and Sewerage Works as a Municipal Utility. R. E. Ward. Tex. W. W. Short School **21:** 50 ('39). Interesting story of how water, light and sewerage plant for Georgetown, Tex., built in 1884 as private plant, was purchased by City in '10 and developed into modern plant now valued at $260,000. To cope with unemployment an arrangement is made whereby over 200 families pay all their utility bills with their own labor.—*O. M. Smith.*

Proceedings of the 21st Annual Texas Water Works and Sewage Short School, Feb. 13–17, 1939. *Edited by* John J. Costley. *Published by Texas State Board of Health, Austin, Texas.* As title indicates, booklet contains registration lists, minutes, reports of committees on licensing, certification of plant operators, sewage research and a list of accomplishments of the water works profession for the yr. Section on water works covers 47 pp. and brief abstracts of papers appear elsewhere. Section on sewage includes 25 papers which are not abstracted by journal. 18 papers are devoted to municipal sanitary status as of January 1, '39, in which for each city is given source of water supply, depth of well, ownership of system and type of treatment; for sewer systems, ownership, type of treatment and wasteway.—*O. M. Smith.*

J. A. Baker et al. Directors and Trustees of Rainbow Lake Outing Club* v. *Happy Valley Water Company. California Railroad Commission Order. Pub. Util. Fort. (P.U.R.) **33:** 126 (June 6, '40). Club leased premises from defendant and diverted water from creek by pipe; defendant removed pipe; club requested order requiring defendant to re-install pipe. Defendant sought dismissal for lack of jurisdiction, maintaining it did not serve the club; that latter obtained water by connecting up to unused pipe. Owner of a water supply has right to make partial or limited dedication and may decline to furnish water to person not within the area of dedication. One claiming right to water utility service must establish that he is beneficiary of public use which owner of water supply is administering and is within district and of class for which dedication is made. Any right to full possession, use, and enjoyment of leased premises where the lessor is water company but has not dedicated its system to public service to leased premises, must be enforced in appropriate proceedings before the civil courts rather than before commission. Complaint dismissed for lack of jurisdiction by commission.—*Samuel A. Evans.*

CHEMISTRY

The Problems of the Chemists. A. SPLITTGERBER. Vom Wasser **13**: 1 ('38). Brief discussion of problems and part played by chemists in water supply and sewage disposal in Germany. Water supplies: ground water 75%, well 13%, surface 6%, impounding res. 6%. In 100 largest water supplies with 172 treatment plants: settling, 17; filtering, 54; iron removal, 74; manganese removal, 18; acidity treatment, 30; and chlorination, 42.—*Willem Rudolfs.*

The Division of Water, Sewage and Sanitation Chemistry. EDWARD BARTOW. Ind. Eng. Chem., News Ed. **17**: 776 (Dec. 20, '39). This division of the Am. Chem. Soc., founded in 1913, has held a no. of symposia either alone or with other divisions of the ACS and has furnished 50 programs of 602 papers for the Society. Much progress in the fields of chemistry covered has occurred since '13. Members of the Division include chemists in water and sewage purification plants, in industries requiring pure water supply and proper waste disposal, for railways, teachers of water chemistry, for state and federal govts. and san. engineers.—*Selma Gottlieb.*

Colloidal Chemistry in Water Treatment. EDWARD S. HOPKINS. Ind. Eng. Chem. **32**: 263 (Feb. '40). Value of pH control in coagulation with alum well established by various workers on both lab. and plant scale. Optimum pH has been shown to be affected by anions present in water, especially sulfates, which shift optimum pH to acid side. Optimum pH varies with nature of water. Sulfate content of floc in zone of min. soly. has led to assignment of formulas such as $Al_2O_3 \cdot SO_3$ to floc, though later observations indicate that sulfate present is adsorbed rather than held in solid solution. Studies indicate that water coagulated at optimum pH settles rapidly when mixed at a definite velocity of flow for a given period of time. Currently favored design is rapid initial mix followed by decreased agitation, thus allowing colloid aggregation into clumps by agitation. Devices such as the hydraulic-jump flume or the flocculator prolong the agitating period and thereby increase clumping action. Colored natural waters have been shown to carry negatively charged colloidal particles, and removal of color by coagulation is probably the most classic pptn. of a true colloid encountered in water purification. With alum, best color removal occurs at about pH 5.4, followed by increase in pH to prevent pipe corrosion, and accompanied by secondary flocculation of hydrated aluminum oxide. Ferric coagulants are also effective in color removal; chlorinated copperas has been shown to be an effective coagulant at pH 4.8, when followed by secondary flocculation with sodium aluminate at pH 6.6. Iron salts are also effective in removing turbidity in the hard waters of the West. The floc is not amphoteric, unlike alum floc. Pptn. is complete at pH values above 3.0, though data of Bartow, Black and Sansbury (not confirmed by Hopkins) indicate best coagulation from pH 4.8 to 7.0 and 8.1 to 9.6. Data of Hopkins indicate optimum pH at 9.4. In Baltimore Water Works, iron and manganese are removed at pH 9.0 with only 0.03 p.p.m. of residual iron. Max. turbidity removal is at same pH. Chlorinated copperas gave compact floc at any pH above 3.5. Analysis of the floc indicated that sulfate was not adsorbed when less than 85 p.p.m. of ferric salts were used. Indications are that sulfate

when adsorbed is present as a "solution link" and stabilizes the soln. at low pH values. Soluble manganese and iron may be removed from water by aeration followed by adsorption in contact bed of coke, pyrolusite or manganese- or iron-coated gravel or stone. Initial pptn. of hydrated oxide in the bed is believed due to stoichiometric reactions but subsequent removal of iron and manganese is thought due to catalytic action of this layer on soluble salts as phenomenon of surface adsorption followed by oxidation.—*Selma Gottlieb.*

New Ideas in the Field of Water Treatment. GRIESSBACH. Melliand-Textilber. **20:** 577 ('39). Certain resins containing acidic groups, e.g., CO_2H, SO_3H, CH_2SO_3H, phenolic OH, can be used to replace cations, e.g., Ca ions, by Na or H ions, while resins containing NH_2 or NH groups are capable of replacing anions, e.g., Cl or SO_4 ions, with OH ions. Combination treatment makes possible removal of 97% of salt ions from water of ordinary hardness. Acidic resins are made from polyhydric phenols, natural tannins, etc., reacted with CH_2O and basic resins from aldehydes condensed with amino bases, especially those of aromatic series. pH intervals, within which ion exchange occurs, are given for no. of resins. See following abstract.—*C. A.*

Treating Water for Use in Dyeing with Ion Exchangers. RICHTER. Melliand-Textilber. **20:** 579 ('39). (Cf. previous abstract.) Resinous ion exchangers are more economical than old-type permutite exchangers because of greater through-put capacity and smaller pressure drop. Elastic properties of resins ensure slower deterioration in use than is case with permutites. Detailed description is given of commercial-scale apparatus using resin ion exchangers for softening water and for removing both anions and cations for special purposes.—*C. A.*

Removal of Copper from Natural Water. B. A. SKOPINTSEV AND M. T. GOLUBEVA. J. Applied Chem. (U.S.S.R.) **12:** 813 ('39). Soluble Cu can accumulate in natural water on treatment with $CuSO_4$. Cu can be removed by treatment with $Al_2(SO_4)_3 \cdot 18H_2O$ (optimum pH 7.0 8.0; however, large amts. of this salt cause an increase of acidity of treated water and decreases Cu removal. Cu can be removed by filtration through sand filter; process has all characteristics of sorption filtration. Washing used sand filter with Cu-free water removes Cu from filter but much more slowly than it is adsorbed. Suspended substances such as clay, sand chalk, activated C and plankton organisms remove some of Cu, but effect is much smaller.—*C. A.*

Decolorization of Water Containing Humic Acid by Means of Sodium Aluminate as a Precipitating Agent. HANS BUCHWALD. Arch. Hyg. Bakt. **122:** 251 ('39). Highly colored water contg. humic acid can be decolorized by filtration through Magno followed by addn. of Na aluminate (30 mg./liter). The Al ppt. is removed by filtration through sand and a completely colorless water results. **Treatment of Humic Acid Waters.** *Ibid.* **122:** 255 ('39). Humic acid-contg. water from Borkum was efficiently decolorized by means of $CaCO_3$ and Fe sulfate.—*C. A.*

Cinderella of the Phosphates. R. E. HALL. Rept. New Engl. Assn. Chem. Teachers **41:** 18 ('39). $NaPO_3$ in acid or alk. soln., and particularly at elevated temp., reverts to orthophosphate, suppresses ionization of many metallic salts in soln., coagulates albumin (ortho- and pyrophosphate do not), has high dispersive action, and is very effective in stabilizing supersatn. of Ca salts, particularly $CaCO_3$. 1 to 2 p.p.m. added to hard bicarbonate water will prevent pptn. of $CaCO_3$, while water contg. 300 p.p.m. of hardness as $CaCO_3$ will not yield ppt. even when treated with Na_2CO_3. After-pptn. in water treated with lime can be avoided by adding 1-2 p.p.m. $NaPO_3$. This "threshold treatment" also largely eliminates deposition of scale in water heaters.—*R. E. Thompson.*

Surface-Active Properties of Hexametaphosphate. G. B. HATCH AND OWEN RICE. Ind. Eng. Chem. **31:** 51 (Jan. '39). Glassy sodium metaphosphate, $(NaPO_3)_x$, called Graham's salt or sodium hexametaphosphate (SMP), possesses definite surface-active properties at solid-aqueous soln. interfaces, and forms soluble complexes with numerous multivalent cations, reducing their concn. so low as practically to eliminate their agglomerating action toward various colloidal systems. Its various industrial uses include prevention of pptn. of Ca soaps or $CaCO_3$ from hard waters. First application of SMP was with ratio of Ca to SMP of approx. 1 to 14; Ca ion concn. is so greatly lowered that it cannot be detected by usual precipitants such as soaps, carbonates, etc. In "threshold treatment" in irrigation waters, no $CaCO_3$ is pptd. upon addn. of 500 p.p.m. of NH_3 to a bicarbonate water with 200 p.p.m. of hardness, using only 2 p.p.m. of SMP, whereas softening would require 1100 p.p.m. of SMP. SMP is also effective in preventing $CaCO_3$ pptn. when CO_3 of bicarbonate waters is moderately increased with other alkali, Na_2CO_3 or heating. SMP dosages of this order, 1 to 5 p.p.m., do not soften water measurably and do not prevent pptn. of Ca soaps, nor $CaCO_3$ pptn. in presence of excess alkali, Na_2CO_3 or heating. "Threshold treatment" prevents $CaCO_3$ scale in cooling systems, heat exchangers, feed-water heaters and boiler feed lines if temp. are not excessive. SMP prevents after-pptn. from lime-soda softened waters, is useful in combating corrosion in water lines since pH can be raised without $CaCO_3$ pptn., and also slowly removes previously formed $CaCO_3$ scale. Lab. tests showed that 2 p.p.m. of SMP is at least as effective as higher dosages in preventing $CaCO_3$ pptn. from a bicarbonate water contg. 600 p.p.m. of hardness on heating to 80°C. for 1 hr. For each temp. studied between 40° and 80°, there is a max. $Ca(HCO_3)_2$ concn. above which $CaCO_3$ pptn. cannot be entirely prevented by 2 p.p.m. of SMP. Concn. of SMP needed for complete protection increases with rising temp. NaCl or Na_2SO_4 up to 200 p.p.m. or moderate amts. of silicate have no adverse effect. SMP treatment inhibits pptn. of $MgCO_3$ but is not very effective against $Mg(OH)_2$. In presence of Ca, SMP is adsorbed on pipe or other metal surface so that inhibiting effect persists briefly after SMP treatment is stopped. In passing "threshold treated" waters over $CaCO_3$ as in filtration of lime-soda softened waters through encrusted sand, pptn. is not immediately prevented, but due to this adsorption, it shortly falls to zero. Sands used with "threshold treated" waters show same lag as noted

above for metal surfaces when SMP is discontinued. Rate of soln. of $CaCO_3$, as from encrusted sand, in distd. water is considerably decreased in presence of SMP, probably due to adsorption of SMP on $CaCO_3$. When cold soln. of $CaCl_2$ equivalent to 400 p.p.m. of $CaCO_3$, contg. 2 p.p.m. of SMP, is treated with equivalent amt. of Na_2CO_3, faintly opalescent soln. is obtd. showing strong Tyndall cone. Soln. is stable in cold for 6 hr. though crystallization is evident after 18 hr. in cold or much more rapidly on heating. In waters contg. 200 p.p.m. of hardness as $CaCO_3$, pH is higher in presence of 2 p.p.m. of SMP, indicating the SMP soln. is supersaturated with respect to $CaCO_3$ and that function of SMP may be to provide increased stabilization for supersaturated state. Deposition upon any $CaCO_3$ nuclei formed is inhibited by adsorption on it of SMP, thus preventing growth beyond colloidal dimensions. Indications are that less CO_2 is lost from SMP treated than from untreated $Ca(HCO_3)_2$ soln. on heating. After 9 mo. treatment of effluent from cold process lime-soda softener with 2 p.p.m. of SMP, 14" line carrying 1400 g.p.m. showed considerable decrease in thickness of scale. Original scale was hard and compact; after treatment it was soft and crumbly or slimy on both sides, and easily detached from metal, possibly due to adsorption of SMP on $CaCO_3$ and metal surfaces. Effect increases with rising temp. and water velocities. Threshold treatment of water with SMP has found wide industrial use during last 2 yr.—*Selma Gottlieb.*

Silicomolybdate Method for Silica. Harold W. Knudson, C. Juday and V. W. Meloche. Ind. Eng. Chem., Anal. Ed. **12:** 270 (May, '40). Study was made of Dienert-and-Wandenbulcke method for rapid detn. of dissolved silica in natural waters of very low phosphate content. Proper pH adjustment is very important with smaller amts. of silica. Though less critical for higher concns., color development falls off rapidly on either side of optimum pH range. Max. color developed within 10 min. and remained unchanged for at least ½ hr. To 100 ml. sample, add 2 ml. of 10% ammonium molybdate soln. Mix and immediately (within 1 min.) acidify to pH 1.6. (Authors used 1 ml. of 4 *N* H_2SO_4 but amt. needed should be predetermined.) After 10 min., compare with standards or read in a photometer from which a standard calibration has been made. Buffered chromate solns., as described by Swank and Mellon, are recommended as standards. Results are better when colors are read in photoelectric colorimeter rather than in Nessler tubes.—*Selma Gottlieb.*

Determination of Sulfate by the Tetrahydroxyquinone Method. H. Lewis Kahler. Ind. Eng. Chem., Anal. Ed. **12:** 266 (May, '40). Sulfite, frequently used in boiler water treatment to remove oxygen, causes high results in detn. of SO_4 by tetrahydroxyquinone (THQ). To eliminate sulfite, boil 25 ml. sample 2 min. with 1.0 ml. of approx. 0.5 *N* HCl. Cool, neutralize with NaOH just to acid side of phenolphthalein, add 25 ml. of ethyl or isopropyl alcohol, then use THQ indicator and titrate as usual with $BaCl_2$. In THQ method, results are best between 8 and 1,000 p.p.m. Endpoint is sharpened by addn. of 1 to 3 ml. of approx. 0.1 *N* $AgNO_3$ before titrating. $AgNO_3$ must be insufficient to react with all Cl present. Sample should be so chosen that max. of 10 ml. of $BaCl_2$ soln. is used (1 ml. = 1 mg. SO_4). If sample is very alkaline,

it should be neutralized roughly with 1 *N* HCl and brought to desired pH with 0.02 *N* HCl or NaOH. Phosphate tolerance can be raised to 150 p.p.m. by diluting 10 ml. sample to 25 ml. followed by neutralization to yellow range of bromocresol green indicator.—*Selma Gottlieb.*

Necessity of Detailed Chemical Analyses for Examinations of the Potability of Water and Hydro-Geological Studies. H. SCHOLLER. Tech. San. Mun. (Fr.) **35**: 15 (Jan.-Feb. '40). Ordinary determinations of chlorides, nitrites and nitrates are insufficient to determine potability. Minimum analyses should include: Mg/Ca, Na/Ca, SO_4/Cl ratios. Complete analyses are necessary to determine type and source, duration of subterranean distance travelled, duration of contact with atmosphere, etc. Long experience in Tunis has shown need for complete analyses to make comparisons of quality. Results can be graphically shown in various ways. Time to discard ancient methods and work on basis of complete analyses, because incomplete analyses make serious hydrogeologic studies absolutely impossible.—*Willem Rudolfs.*

Determination of the pH of Water. F. R. MCCRUMB. W. W. and Sew. **86**: 403 (Oct. '39). In electrometric detn., 3 electrodes may be used: hydrogen, quinhydrone, or glass. First is not practical for water, second is satisfactory only for well-buffered waters, and third seems fairly satisfactory, although it possesses some disadvantages. Colorimetric methods are most commonly used. Indicator soln. pH should be measured and kept close to midpoint of range for which indicator is used. Indicator soln. may change pH enough to invalidate test results. Data on this effect are included. Salt concn. in water are lower than those employed in making color standards, and this frequently causes errors as high as 0.2 pH. Absorption of CO_2 from air frequently causes considerable error, particularly with distilled waters. Stoppering of tube with thumb during mixing of indicator and sample is abhorred. CO_2 concn. above 2.0 p.p.m. may introduce errors.—*H. E. Hudson, Jr.*

The Calculation of the Distribution of Carbon Dioxide between Water and Steam. A. M. AMOROSI AND J. R. MCDERMIT. Proc. A.S.T.M. **39**: 1204 ('39). Significance of dissolved CO_2 in water and principles of apparatus for removal of dissolved gases are discussed. Behavior of CO_2 is differentiated from behavior of O_2 in de-aeration. Effect of temp. and pH on ionization is calculated indicating distribution of CO_2 as H_2CO_3, HCO^-_3 and $CO^=_3$. Concluded that on high pH waters, CO_2 as gas can readily be removed since amount present as such is very small. On waters of lower pH significant removal can be obtained but solubility and change in form relationships intervene to make actual results less than expected results.—*T. E. Larson.*

The pH, Dissolved Iron Concentration and Solid Product Resulting from the Reaction between Iron and Pure Water at Room Temperature. R. C. COREY AND T. J. FINNEGAN. Proc. A.S.T.M. **39**: 1242 ('39). Reaction between iron and pure water at room temp. was carried out in iron vessels with water free from O_2 and CO_2. This insured absence of impurities from glass and absence of O_2 and CO_2 factor. Solution reached a pH of 8.3 and contained 0.2 p.p.m. dissolved iron. Solid product was predominantly Fe_3O_4.—*T. E. Larson.*

Simplified Calculation of the Carbon Dioxide in Equilibrium in Natural Water and Some Practical Applications. JOSEF TREGL. Gas-u. Wasserfach **82**: 715 (Oct. 21, '39). Tillmans Calcn. of equilibrium in CO_2 is modified. Amt. of bond CO_2 is assumed to be proportional to carbonate hardness and amt. of calcium oxide is taken as being proportional to total hardness. Amount of CO_2 in equilibrium in sample is then calcd. and expressed in units (as used in U. S.) at temp. of 17°C. by:

$$CO_2 \text{ in equilibrium in p.p.m.} = 2.14 \times 10^{-6}A^2B$$
$$\text{or} = 0.0107\ C^2D$$

where A = carbonate hardness in p.p.m., B = total hardness in p.p.m., C = carbonate hardness in g.p.g., D = total hardness in g.p.g. If then CO_2 in equilibrium calcd. is more than free CO_2 found, water contains no aggressive CO_2. On other hand, any excess of free CO_2 found beyond that calcd. has to be considered as aggressive CO_2. Increase in carbonate hardness required to neutralize this aggressive CO_2 can be calcd. as well as requirements on limestone, Magno-mass or hydrated lime by considering that an increase of 1 p.p.m. in carbonate hardness causes reduction of .44 p.p.m. of aggressive CO_2 and this requires in reagents per 1000 gal. of water treated either 3.8 grams $CaCO_3$, 2.8 grams Magno-mass or 2.9 grams $Ca(OH)_3$. Direct calcn. is impractical, as it requires soln. of a cubic equation and is therefore better done by a stepwise calcn. with final interpolation. The calcn. can also be made for hot water, whereby other constants have to be used. Recalcd. in our system for units in p.p.m. these are given as: for 40°C., 4.1×10^{-6}; for 60°C., 7.1×10^{-6}; and for 80°C., 13.1×10^{-6}. The max. carbonate hardness for normal temp. and pressure is calcd. from soly. of CO_2 in water to 950 p.p.m. It is shown that if non-carbonate hardness is present besides carbonate hardness, the max. total hardness is > that for carbonate hardness.—*Max Suter.*

The Volumetric Method of Andrews for the Determination of Sulfuric Acid, as Used for Water with Varying Hardness. Grigor'eva Lab. Prakt. (U.S.S.R.) **2-3:** 29 ('39). Samples of both natural and artificially prepd. water were investigated in testing method of A l.c. for detg. sulfuric acid. Hardness varied from 1 to 124 German deg. and amt. of sulfuric acid from 2 to 3710 mg. per liter of water. K l.c. method was employed for pptn. of sulfate both from artificial and from natural waters. Barium chromate was prepd. according to method of B. l.c.; 1 ml. of solution of barium chromate ppt. 100 mg. of sulfate. For titration, approx. 1 ml. of thiosulfate solution was used for each mg. of sulfate. It was concluded that this volumetric method for detn. of sulfate is satisfactory for waters of different hardness. Best results are obtained by pptg. from 100 ml. of water contg. 10-20 mg. of sulfate, and using 10 mg. of barium chromate for each 10 mg. of sulfate. Exptl. error which depends on chem. compn. of substances in the water, is ± 2%.—*W. P. R.*

A Comparison of Different Methods for Determining Small Amounts of Lead. M. V. NEUSTRUEVA. Trav. inst. état radium. (U.S.S.R.) **4:** 304 ('38). Poor results are obtained when small amts. of lead are determined by electrolysis, weighing lead dioxide pptd. on anode, or by using radium *D* as an indicator.

Results are satisfactory if lead is detd. colorimetrically using sodium sulfide or tetramethyldiaminodiphenylmethane; with second substance method is complex and, in very dilute soln. color fades rapidly.—*W. P. R.*

Color Tests for Chlorine, Ozone, and Hypochlorites. A. T. MASTERMAN. Analyst. **64:** 492 ('39). When ozone gas is passed into a soln. of "methane base" (4:4: tetramethyldiaminodiphenylmethane) in carbon tetrachloride, alcohol, or acetone, liquid turns violet in color; changing gradually to ruby red. When chlorine is used instead of ozone, colors produced change from blue, through green, to yellow; color is finally bleached. Aq. soln. of hypochlorites give at first same colors as chlorine. Hypochlorites prepared electrolytically may produce violet to rose colors after production of chlorine colors; this only occurs after standing for some time and is probably due to the presence of ozone formed during electrolysis. Chemically prepared hypochlorites do not give blue-green colors in aq. soln. They are capable, however, of giving blue-green colors after prolonged standing; these colors may be somewhat intensified and their production accelerated by addition of sodium chloride. In conditions under which hypochlorites can oxidize methane base, colors produced are violet or red; these conditions are the presence of excess alc. with a min. amt. of water or presence of excess methane base.—*W. P. R.*

Chlorine. The Heat Capacity, Vapor Pressure, Heats of Fusion and Vaporization, and Entropy. W. F. GIAUQUE AND T. M. POWELL. Jour. Am. Chem. Soc. **61:** 1970 (Aug. '39). Melting and boiling points were found to be —101°C. and 34°C. respectively. Heat of fusion is 1531 cal. per mole, and heat of vaporization at boiling point is 4878 cal. per mole. Vapor pressure is represented from triple point 172.12 to 240.05°K by the equation:

$$\log_{10} P_{(\text{cm. Hg})} = \frac{-1414.8}{T} - 0.01206T + 1.34 \times 10^{-5}\,T^2 + 9.91635$$

Results by calculating from equation agree closely with observed values. Entropy and heat capac. values were also detd.—*Selma Gottlieb.*

Determination of Organically Bound Arsenic in Potable Waters. JÁNOS CSABAY AND ISTVÁN TANAY. Ber. ungar. pharm. Ges. **15:** 83 ('39). To det. org. arsenic compds. which may have entered the water from war gases, add to 100 c.c. water, 10 c.c. concd. H_2O_2 and 5 c.c. concd. H_2SO_3, evap. on water bath to 30 c.c., pour soln. into 10-c.c. Kjeldahl flask and wash dish twice with 2.5-c.c. portions of H_2O_2. Cautiously heat flask. After gas development wash dish with four 2.5-c.c. portions of H_2O_2, after gas evolution stops add 2 c.c. fuming HNO_3 and heat again cautiously. Cool, add 5 c.c. H_2O_2, heat carefully, add again 5-c.c. H_2O_2 and boil flask for 10 min. to decomp. excess peroxide. Reduce the soln. according to Schulek-Villecz (cf. *Ibid.* **11:** 2 ('35)), cool flask, add 30 c.c. water, 0.2 gram KBr and 2 drops of an alc. 0.5% soln. of α-naphthoflavone. Titrate with 0.1 *N* $KBrO_3$ to a yellowish brown. Method can be used for microdetns. in similar manner with 0.01 *N* $KBrO_3$ for final titration. The soly. of *diphenylchloroarsine*, detd. by this method, is 0.06 grams/liter at 20°.—*C. A.*

First Report of the Sub-Committee on the Determination of Arsenic Lead, and Other Poisonous Metals in Food-Coloring Materials to the Standing Committee on the Uniformity of Analytical Methods (Society of Public Analysts). I. The Determination of Arsenic. T. Callan and S. G. Clifford. Analyst. (Br.) **55:** 102 ('30). Arsenic in dyestuffs is determined by distn. with sulfuric acid and a mixture made up of 5 gram sodium chloride, 0.5 gram hydrazine sulfate, and 0.02 gram potassium bromide; the distillate, collected in dilute nitric acid, is evaporated to dryness, and arsenic is determined by the G. l.c. method. More accurate results are obtained by oxidizing coloring matter before carrying out distn. **Second Report. II. The Determination of Lead.** T. Callan and S. G. Clifford. *Ibid.* **60:** 541 ('35). Discusses several methods for detn. of lead in soln. remaining after detn. of arsenic (see Part I). Relative advantages of separation of lead by electrolysis, by dithizone, as sulfate, and as sulfide are dealt with in detail. Colorimetric detn. of lead as lead sulfide formed by adding sodium sulfide is recommended. Tables summarizing percentage error in detn. of known amounts of lead, and showing variations in results obtained on analysis of same material by different observers. Exptl. details of recommended method are given. **Third Report. III. The Determination of Copper.** T. Macara, and S. G. Clifford. *Ibid.* **64:** 339 ('39). Copper remains in soln. after separation of lead as sulfate (see Part II). Copper is detd. colorimetrically as the golden brown complex compound formed on addition of sodium diethyldithiocarbamate. Interference by metals such as iron or aluminium is suppressed by addn. of ammonium citrate and ammonia. If necessary colored complex may be determined after extn. in org. solvent. Details given for making detn. analyses by this method of a solution containing 24.2 p.p.m. copper, and iron, aluminium, tin, lead, zinc, calcium, and phosphates, gave variations from true copper content of − 1.5 to + 2.2 p.p.m.—*W. P. R.*

Determination of Fluorine in Wine. H. G. Rempel. Ind. Eng. Chem., Anal. Ed. **11:** 378 (Jul. '39). Willard and Winter method is modified. A single steam distn. (of wine ash) over perchloric acid in a 125 ml. distg. flask yielded a distillate free from interfering ions. The 200 to 250 ml. distillate was made alkaline to phenolphthalein with satd. Na_2CO_3 and evapd. down to about 10 ml. on a hot plate at temp. not over 85°C. The concd. distillate was neutralized with 1:20 HCl to discharge phenolphthalein color, 1 ml. of 0.05% aq. soln. of sodium alizarin sulfonate added and acidity carefully adjusted with approx. 0.01 N HCl to golden yellow indicator color. Soln. was made up to 100 ml., divided into 2 equal parts and 50 ml. of 95% ethyl alcohol (pH 5.0) added to each aliquot. Solution was titrated with 0.01 N thorium nitrate to endpoint produced by adding 0.1 ml. of 0.01 N thorium nitrate to a soln. contg. no F and 0.5 ml. of indicator. Vol. of thorium nitrate used — 0.1 ml. for indicator blank corresponds to F present. As little as 0.01 mg. of F can be detd. in aliquot.—*Selma Gottlieb.*

A New Method for the Colorimetric Determination of Fluorides. V. P. Shvedov. Lab. Prakt (U.S.S.R.) **2-3:** 22 ('39). Sodium alizarinsulfonate, purpurin, anthrapurpurin, flavorpurpurin, anthagallol, alizarincyanine and

cerulein were tested as indicators in colorimetric determination of fluorides. Most sensitive indicator was 1,2,4,5,8-alizarincyanine; this had a max. sensitivity of 0.01 mg. fluoride per liter and an average of 0.02 mg. fluoride per liter. At room temp. reaction between fluoride and alizarincyanine takes place in 2-3 hr. With 100 mg. sample, addition of 5 ml. of indicator and 10 ml. of $\frac{1}{12}$ N hydrochloric acid gives max. sensitivity. With increasing concn. of fluoride color changes from blue to violet. Effect of potassium, sodium, calcium, magnesium, chloride, bromide, and nitrate ions is slight, but color is considerably affected by aluminium, sulfate, phosphate, arsenite, and arsenate ions.—*W. P. R.*

Colorimetric Determination of Fluorine with Ferron. JOSEPH J. FAHEY. Ind. Eng. Chem., Anal. Ed. **11:** 362 (Jul. '39). Ferron (7-iodo-8- hydroxyquinoline-5-sulfonic acid), previously used as colorimetric reagent for iron, can be used for detn. of F, including 1 p.p.m. or more of F in natural waters. Reagent contains 90 ml. of satd. water soln. of ferron, 10 ml. of soln., 2 N in HCl and 0.1 N in ferric chloride, and 100 ml. of distd. water. Pipette 25 ml. of sample or aliquot into a 50 ml. beaker; and into another beaker, 25 ml. of soln. having same pH and NaCl content as sample. To each add 2.00 ml. of ferron-iron reagent. Colors are compared in a colorimeter and 0.02 N NaF added to comparison soln. until colors match, equal vol. of distd. water being added to sample. Samples and comparison soln. were brought to pH 4.2 before adding ferron reagent. Method might be made sensitive enough for smaller quantities of F by using 0.005 N NaF, and reagent contg. less HCl.—*Selma Gottlieb.*

Development and Use of the Bone Filter for Removing Fluorine from Drinking Water. H. V. SMITH AND W. B. DAVEY. Ariz. Agr. Expt. Sta., Tech. Bull. 81 ('39) p. 249. Sanchis method for detn. of F gives only approx. results. PO_4 ions cause low results and SO_4 ions cause high results. Bone is prepd. for use in removing F from drinking water by boiling with alkali until it has lost its flinty characteristics and has become snow white in appearance, washing out excess alkali with H_2O, and neutralizing with dilute HCl. Bone can be regenerated after use by similar treatment. Calcination of bone at 400-600° for 10 min., followed by 10-min. acid treatment, yields a better bone product for F-removal process. Bone prepd. in this way will not putrefy as do some of alkali-acid treated products. A bone fragment size of 40-60 mesh was most desirable. Period of contact directly affects percentage removal of F. pH of water had little effect unless it was too alkaline. Presence of salts other than fluorides had little effect on F removal. Double treatment of water with 2 portions of bone was no better than single treatment with twice as much bone. Mechanism by which bone removes F from water appears to be through formation of solid solns. rather than simple anion exchanges as was at first supposed. Other phosphates were tried in filters but none worked as well as bone. Bone is about twice as effective as the commercial $Ca_3(PO_4)_2$, Defluorite. It is entirely practical to remove excess F from drinking water supplies by means of bone filters. 37 refs.—*C. A.*

Estimation of Boron by a Modified Flame Test. H. C. Weber and R. D. Jacobson. Ind. Eng. Chem.-Anal. Ed. **10:** 273 (May '38). Air passed at 150 ml. per min. through $CaCl_2$ drying tube and flowmeter into a test tube contg. 6 cc. of methyl alcohol and 1 ml. of concd. H_2SO_4 with the boron sample previously evaporated almost to dryness. Air carrying methyl alcohol and methyl borate passes out of test tube through nozzle and is ignited at jet by fan-shaped Bunsen flame at right angles to jet. For quantity of boron from 0.02 to 0.1 mg., duration of green color in flame is almost straight line function of boron content, time averaging 79 to 362 seconds respectively.—*Selma Gottlieb.*

Systematic Investigation of Waters for Bromine. Ernesto E. J. Bachmann and Francisco C. T. Pertini. Bol. Obras Sanitarias Nacion (Buenos Aires) **3:** 396 ('39). Ten ml. of concd. water is mixed in a wash bottle with a H_2SO_4 soln. of CrO_3; slow current of air is aspirated through liquid and through disk of filter paper (S. & S. thick paper for spot tests) impregnated with a 0.2% soln. of fluorescein; about 10 min. is required to sweep out liberated Br. Paper disk is then exposed to fumes of NH_3 to develop color of NH_4 salt of eosin which is compared with previously prepd. scale. Iodine and nitrate do not interfere, nor does Cl in amts. not exceeding 100 mg./l.; in that case Br is first absorbed in soln. of 1% Na_2SO_3 + 0.2% Na_2CO_3 which is concd. and treated as above. No Br was found in water of Plata, Uruguay or other large streams; some well waters contained up to 0.8 mg./l.—*C. A.*

Determination of Iodide in Mineral Waters Containing Bromide and Chloride. Ernst Müller and Walter Stump. Z. Anal. Chem. (Ger.) **118:** 90 ('39). When small quantities of I^- in mineral waters are detd. by measuring color produced in solvent immiscible with water, after treatment with KNO_2 + HCl, results are usually low because some of the I^- is oxidized to IO_3^-. Presence of considerable Cl^- or Br^- seems to catalyze oxidation to HIO_3. If, however, quantities of KNO_2 and HCl used are restricted, oxidation to HIO_3 is avoided, although slight excess of oxidizer or acid may cause incomplete oxidation of I^- to I_2. It is proposed, therefore, to carry out a rough detn. of I^- in preliminary expt. and then prepare a soln. contg. approx. same quantities and determine I^- content of this soln. with quantities of KNO_2 and HCl in very slight excess. From this analysis, with a known quantity of I^-, an empirical factor can be obtained to be used in analysis.—*C. A.*

Mercurous Perchlorate as a Volumetric Reagent for Chlorides and Bromides. W. Pugh. J. Chem. Soc. (Br.) '37: 1824. $HgClO_4$ reacts with halide very much the same as $AgNO_3$ and can be used as a volumetric reagent. Good results were obtained in determining Cl^- in samples of $PbCl_2$, $BaCl_2$ and sea water. Solution for analysis, which had vol. of about 100 ml. and halide concn. of not over 0.04 *N*, was boiled and treated with 5% $Pb(NO_3)_2$ if SO_4^{--} was present: a drop of bromophenol blue indicator soln. was then added and soln. was neutralized with NH_3 or soda until a faint blue color was obtained. Just before end point in titrating with 0.01 or 0.05 *N* $HgClO_4$, 0.5 ml. more of indicator soln. was added and titration continued until color of soln. changed to lilac. The bromophenol blue acts as an adsorption indicator.—*C. A.*

Colorimetric Determination of Manganese with Periodate. J. P. MEHLIG. Ind. Eng. Chem. Anal. Ed. **11:** 274 (May '39). Using photoelectric recording spectrophotometer, study was made of detn. of Mn with periodate. Color system follows Beer's law at least up to 20 p.p.m. of Mn. Increase in acidity or substitution of HNO_3 or H_3PO_4 for H_2SO_4 does not affect color except with high concns. of HNO_3. Increasing amt. of K periodate has no appreciable effect on color. Color is stable in diffuse light for at least two months. Effect of 56 common ions was studied; reducing ions affect color unless sufficient excess of periodate is used. If cations are present which are pptd. by H_2SO_4, H_3PO_4 can be substd. H_3PO_4 can also be used to decolorize solns. contg. Fe. All Fe should be oxidized to ferric state to avoid liberation of I from periodate by ferrous ion. Effect of 400 p.p.m. of Al, Ca or Mg or 620 p.p.m. of Na is negligible. Colorimetric periodate method is a most satisfactory one with few limitations.—*Selma Gottlieb.*

Formaldoxime as Color Reagent for Certain Metals; Manganese Determination in Water and Iron. G. H. WAGENAAR. Pharm. Weekblad **75:** 641 ('38). After oxidation and removal of Fe, Mn is detd. in 10 ml. water colorimetrically by addn. of 1 drop of formaldoxime reagent (trioxymethylene 3, $NH_2OH \cdot HCl$ 7 and water 15 parts) and 4 drops 4 *N* NaOH. Sensitivity of reaction is 0.08 mg. per liter. Turbidity from CaO or MgO is obviated by adding NH_4Cl.—*C. A.*

The Production of Active Carbon from Bituminous Coal. R. H. O. J. Franklin Inst. **227:** 145 (Jan. '39). Technical Paper No. 47, of the British War Office, describes investigation of this subject with view to producing carbon for gas respirators. Shows that at least one type of carbon of the highest quality can be made, and there is sufficient promise in the preparation of other types to justify further research. It is considered that the plant available in the carbonizing industries could be adapted for this purpose. Activated coke is prepared by carbonizing carefully selected coals of the hard durain type which do not intumesce on carbonization at 500°C. The process consists of carbonizing the graded coal below 500°C. and treating the resulting coke with steam or steam and air in a continuous-vertical retort at about 950°C. During this process about 75% of the coke is gasified, and a 25% yield of activated coke is obtained, which, by crushing and grading, can be made into graded activated carbon. When graded to the correct size carbon equal to a first grade commercial carbon for gas respirators can be made. About 70% of this material can be made from activated coke. Further research and treatment of other sizes may be necessary for other uses.—*Ed.*

Application of a Micro-Photometric Test for Aluminum in the Water of the Federal Capital (Buenos Aires, Argentine Republic). C. J. PEREYRA. Bol. Obras Sanitarias Nacion (Buenos Aires) **3:** 389 ('39). Author reviews literature concerning methods for detn. of small amt. of aluminum in water and describes in detail apparatus and procedure for detn. of aluminum by microphotometric method. Method depends on measurement of opacity of soln. contg. aluminum compounds (10^{-5} to 10^{-7} gram aluminum per liter of water) to which a solution of cupferron has been added; a colloidal ppt. forms when

pH value of liquid is maintained at 2–5; absorption of light by turbid liquid obeys Beer's law. In carrying out test a 200-ml. sample of water is evapd. to dryness with 10 ml. of concd. hydrochloric acid; residue is evapd. with 5 ml. concd. hydrochloric acid and mass is digested for 1–2 min. with 10 ml. N hydrochloric acid before filtration. Filtrate is collected in a 50-ml. measuring flask to which are added 7.5 ml. of a 2 N soln. of sodium acetate and sufficient disd. water to make up vol. to 50 ml.; 1 ml. of 5% aq. soln. of cupferron is added and after 30 min. liquid is examined in a Zeiss-Pulfrich photometer. Correction must be applied when traces of iron are present in sample. Tables of results of analyses of water supplies of Buenos Aires are given. In '38 mean concn. of aluminum in water as detd. gravimetrically was 0.7 mg. per liter; above method gave values of 0.30–0.48 mg. per liter in samples from different points in water supply system.—*W. P. R.*

Determination of Total Dissolved Solids in Water by Electrical Conductivity. H. Gustafson and A. S. Behrman. Ind. Eng. Chem., Anal. Ed. **11:** 355 (Jul. '39). Detn. of total dissolved solids (TDS) usually serves to check sum of constituents as individually detd. or calcd. Standard method of detg. TDS (by evapn. of measured vol. of water and drying residue at specified temp. to constant wt.) is laborious, time-consuming and frequently uncertain. Elec. conductance has been used for many years as rough index of TDS to detect condenser leakage, etc. Attempts to use conductance for quantitatively accurate measurements have failed because of different conductance values of different constituents in the waters. Conductance values were detd. for dilute solns. of usual mineral constituents of water. Conductances of synthetic waters from mixt. of these solns. checked with values calcd. from individual conductances with max. deviation of 4%. For natural waters, conductance values detd. checked satisfactorily with those calcd. from results of mineral analysis, and method has been used with consistent accuracy in over 1000 samples. From TDS as calcd. from mineral analysis, sample is diluted to conc. of 40 to 60 p.p.m. as $CaCO_3$, and air bubbled through to remove free CO_2. Temp. should be 25° ± 3°C. Conductivity is detd. in any suitable manner. Authors use slide wire of a Leeds & Northrup potentiometer, a resistance box, a microphone hummer as a source of high frequency current, and a dip-type conductivity cell. Usual correction of 2% per degree temp. rise is applied to bring results to 25°C. Procedure is useful, e.g. in checking results of water treatment by hydrogen zeolite and anion exchangers. Noncarbonate solids can be calcd. from conductivity and alkalinity detn. alone, with saving of several hours of time:

$$\frac{\text{Conductivity } (\times 10^6) - (\text{alkalinity} \times 0.186)}{0.25} = \text{noncarbonate solids as } CaCO_3$$

—*Selma Gottlieb.*